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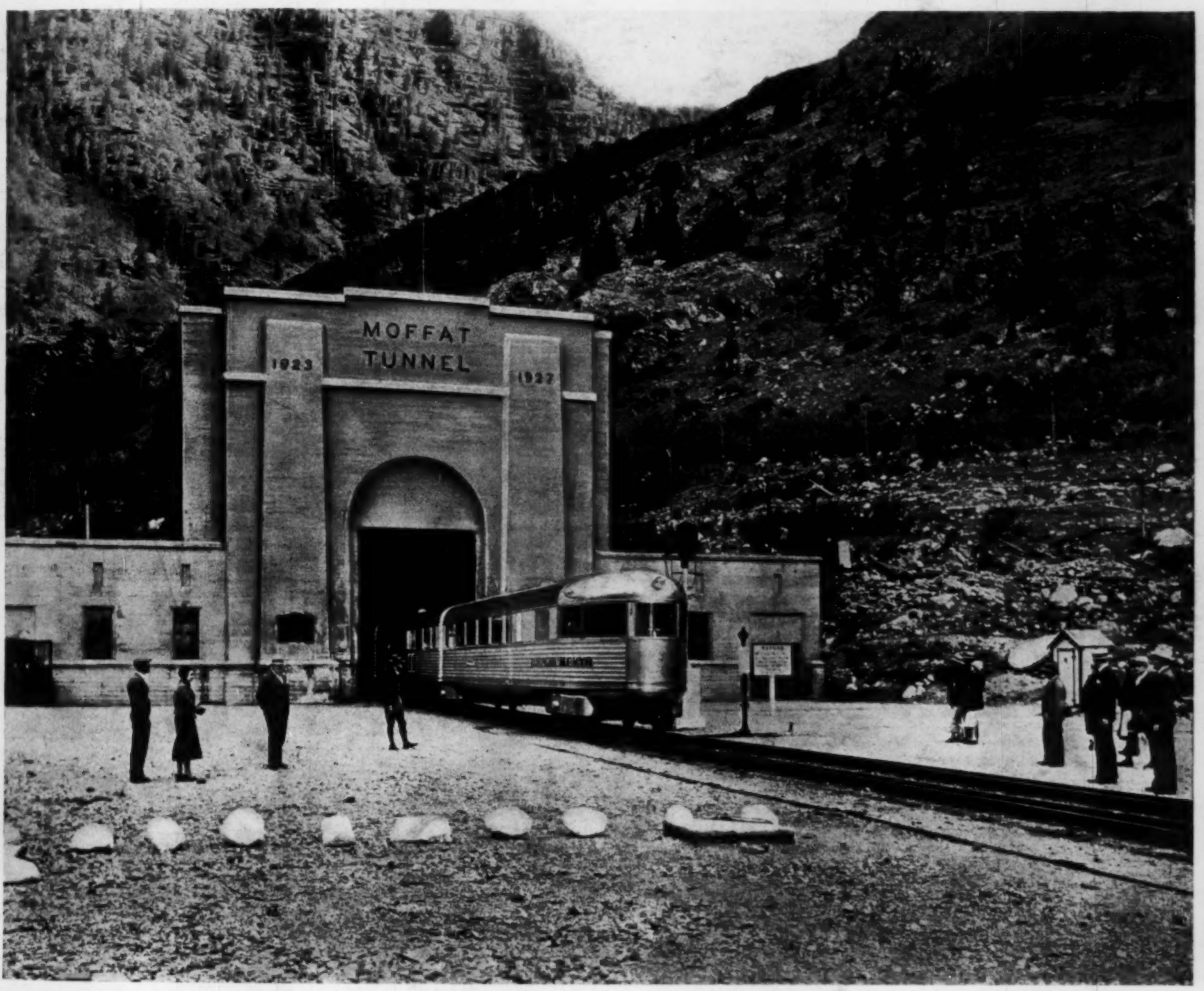

The Burlington Zephyr Entering the East Portal of Moffat Tunnel, June 16, 1934

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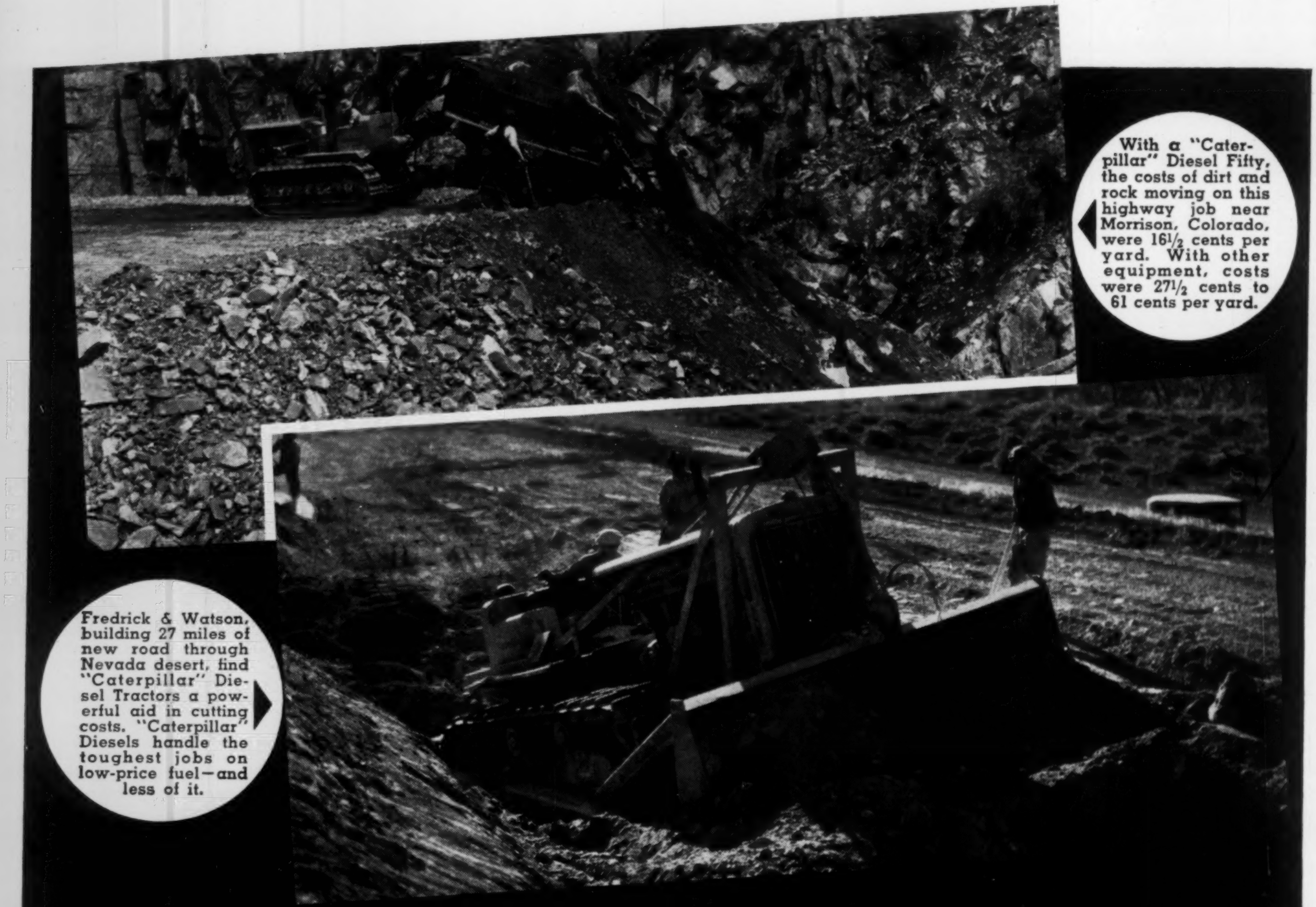


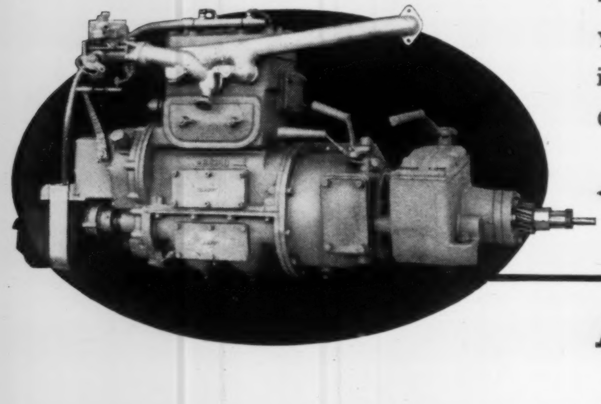
With a "Caterpillar" Diesel Fifty, the costs of dirt and rock moving on this highway job near Morrison, Colorado, were 16½ cents per yard. With other equipment, costs were 27½ cents to 61 cents per yard.

Fredrick & Watson, building 27 miles of new road through Nevada desert, find "Caterpillar" Diesel Tractors a powerful aid in cutting costs. "Caterpillar" Diesels handle the toughest jobs on low-price fuel—and less of it.

"THE 'CATERPILLAR' DIESEL IS THE GREATEST ENGINE I'VE EVER SEEN"

● ● says J. H. Argerter, who operates the Moniteau County, Missouri, "Caterpillar" Diesel Tractor. He adds: "I have worked with many engines since 1920 and this Diesel tractor is beyond a doubt the best of all."

"No expense, no delay"—"Fuel costs below our expectations or even hopes"—"Not a minute's time lost"—"Lots of power, and the smoothest we've ever known"—"Simpler than a gas tractor, and better in every way." . . . Those are typical expressions from "Caterpillar" Diesel owners. Some of them bought on confidence in "Caterpillar" superiority—some on study of "Caterpillar" design and construction—now they have the proof of performance. Study the "Caterpillar" Diesel, or talk to users—either way you'll come to the conclusion that the "Caterpillar" Diesel is the surest, safest investment in power equipment. Caterpillar Tractor Co., Peoria, Illinois, U. S. A.



◀ Mr. R. G. Warren, Supervisor, of Hampton, N. Y., says: "The starting system can't be beat—this system (gasoline starting engine) does away with costly up-keep of batteries and is always a sure starter."

AMERICA GOES DIESEL

Among Our Writers

Ralph Budd has been continuously in railroad work for 35 years. He was engineer of maintenance and chief engineer of the Panama Railroad from 1906 to 1909. After making a number of reconnaissances in Oregon and Washington, he became chief engineer of the Spokane, Portland and Seattle Railway System. In 1913 he entered the service of the Great Northern Railway as chief engineer and from 1919 to 1932 served as president of the system. It was during this time that the line over the Cascade Mountains was electrified and the Cascade Tunnel built. In 1930 he reported to the Soviet Government on the rehabilitation of the railroads in Russia. It was due to his vision that the Burlington System's Zephyr was built.

Karl R. Kennison is a graduate of Colby College and of the Massachusetts Institute of Technology. For a number of years he was principal assistant to the late John R. Freeman on water supply and water power projects, including the Hetch Hetchy project for San Francisco. Later he was engaged on the new water supply project for Providence, R.I., and was chief engineer of the South Essex Sewerage Board. After maintaining a private consulting practice for six years, in 1926 he took up his present work with the Metropolitan District Water Supply Commission of Boston.

H. A. Van Norman has specialized in engineering construction. In 1904 and 1905 he was superintendent of construction for the Los Angeles Gas and Electric Company. From 1906 to its completion in 1923, he was engaged on the construction of the Los Angeles Aqueduct, first in Owens Valley, then on the Mojave Division, and finally in charge of all construction. He built the Hyperion outfall sewer for Los Angeles and was for a time city engineer. Returning to the Bureau of Water Works and Supply of Los Angeles in 1925, he became assistant chief engineer, and in 1928, chief engineer and general manager. The Bureau has recently announced the beginning of construction on an extension of the Los Angeles Aqueduct to the Mono Basin.

E. W. Seemann was field engineer for the Portland Cement Association from 1928 to 1931. Since that time he has been with the Merritt-Chapman and Whitney Corporation as construction engineer, first on Lock No. 15 at Rock Island, Ill.; next on estimating and designing in the Chicago office; then on the construction of the water intake at Wilmette, Ill.; and at present on the construction of Dam No. 5 on the Upper Mississippi River.

Raymond A. Hill is a graduate of the University of Michigan, holding the degree of Civil Engineer. His early experience was on various projects of the U. S. Reclamation Service. During the World War he served in the Corps of Engineers in the A.E.F. Since 1919, Captain Hill has been a member of a consulting engineering firm. He has been widely engaged in the investigation of physical features and financial aspects of irrigation, power, and domestic water supply projects in the western United States and Mexico, and has advised on a variety of special engineering problems, one of which is described in this issue.

Mason J. Young attended the Command and General Staff School, Fort Leavenworth, Kans., in 1929 and 1930. Assigned to duty in the Philadelphia District, Major Young had charge of the preparation of the report on the utilization of the Delaware River. After four years of service there he was assigned in June 1934 to Providence, R.I., where he is District Engineer for the Corps of Engineers.

Searcy B. Slack, a graduate in Civil Engineering of the University of Georgia and of Harvard University, has this year taken up the private practice of highway and bridge engineering after 22 years of varied experience in this field of work. He was for 13 years in charge of bridge work and research for the State Highway Board of Georgia. During that time a large number of important bridges were built in Georgia, and testing and research work was carried on in many phases of highway construction.

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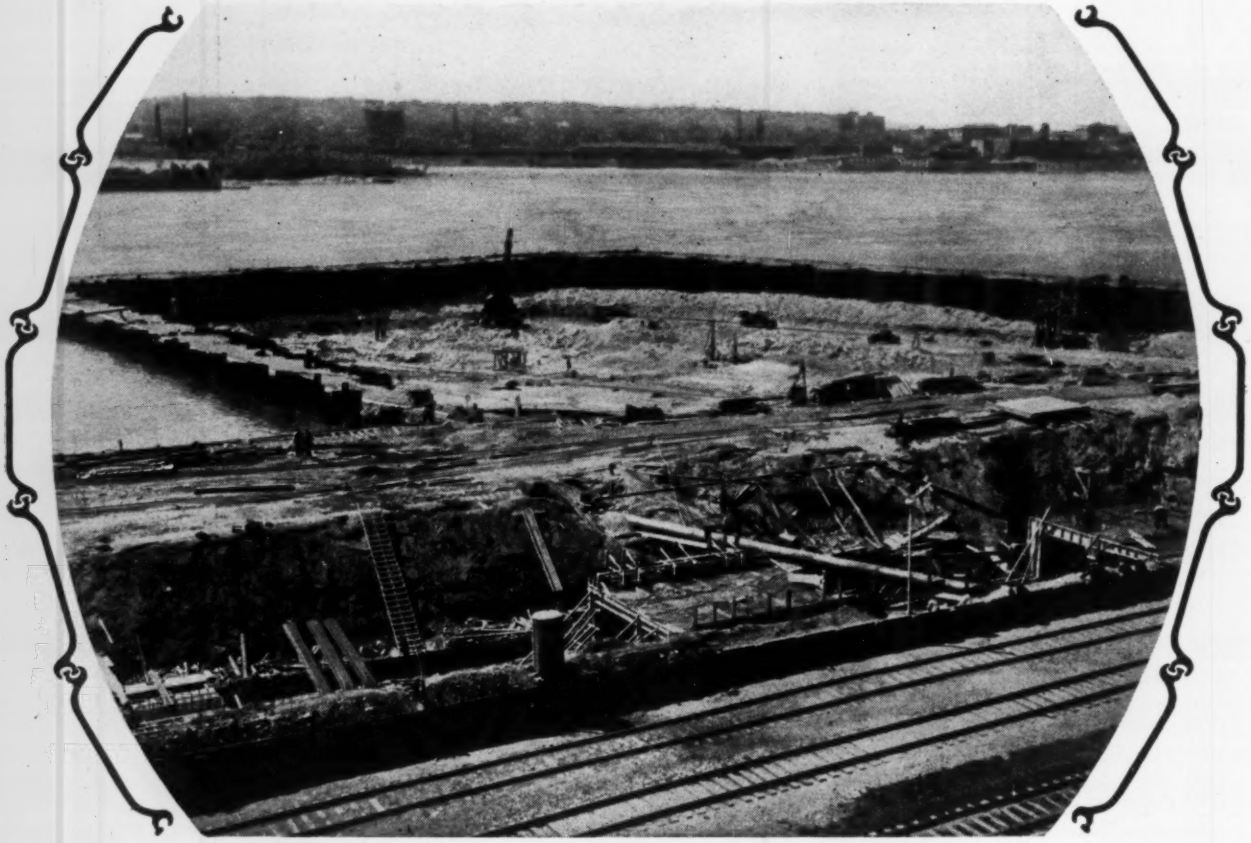

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August 1934

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Number 8

The Burlington Zephyr

Its Design and Construction, and Its Place in the Operation of the System

By Ralph Budd
Member American Society of Civil Engineers
President, Chicago, Burlington and Quincy Railroad, Chicago, Ill.

POWERED with an 8-cylinder, 2-cycle Diesel engine weighing scarcely 21 lb per developed horsepower, the Burlington System's light-weight streamlined passenger train, Zephyr, ushers in a new era in railroad passenger transportation. It is built of a chrome nickel alloy steel and equipped with every conceivable facility for the comfort and convenience of passengers and is frankly a bold attempt to attract back to the railroads traffic that has been deserting them at an alarming rate. Although not yet in regular service, the train has been tested over 15,000 miles of rails from coast to coast. On a non-stop run from Denver to Chicago in May 1934, it covered 1,015 miles at an average speed of 77.6 mph to break all existing records. At one point on the run it attained a speed of 107 mph. How far this and other experiments with high-speed, light-weight trains will be carried by the railroads of the country will depend on the extent to which passenger traffic is retrieved by the new equipment. The possibilities from the mechanical and operating standpoint indicate revolutionary departures from present standard practice.

SINCE 1922 the decline in passenger traffic has been one of the most spectacular features of railway statistics and may account for more of the present low state of railway net income than any other single item. This is particularly true since the reduction in passenger-train miles has not been nearly so great as the loss in passenger revenue. The data in Table I bring out the pertinent facts strikingly.

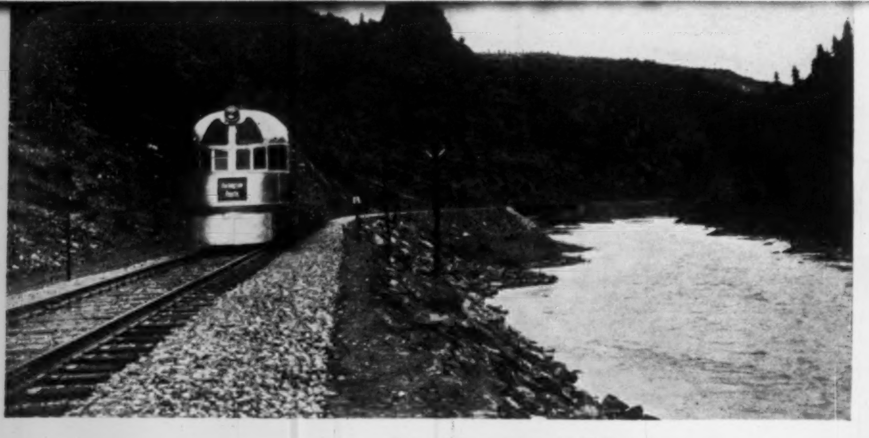
Table I. Number of Passengers Carried on Class I Railways of the Nation One Mile, and Revenue Therefrom

Year	Number Passengers Carried One Mile	Revenue	Revenue per Passenger Mile (Cents)	Passenger-Train Miles
1920	46,848,667,987	$1,286,613,273	2.745	561,633,392
1921	37,312,585,966	1,151,770,842	3.086	554,804,976
1922	35,469,961,582	1,074,108,060	3.027	541,275,421
1923	37,956,594,827	1,145,698,579	3.018	560,980,411
1924	36,090,886,478	1,075,039,219	2.978	566,012,544
1925	35,950,222,811	1,056,395,303	2.938	569,764,884
1926	35,477,524,581	1,041,816,167	2.936	573,626,649
1927	33,649,706,115	974,950,863	2.896	568,537,968
1928	31,601,341,798	901,018,801	2.850	561,599,590
1929	31,074,134,542	872,466,361	2.808	560,692,022
1930	26,814,824,535	728,487,762	2.717	538,709,206
1931	21,894,420,536	550,249,848	2.513	480,367,309
1932	16,971,044,205	376,538,841	2.219	414,383,043
1933	16,345,047,000	329,011,919	2.013	377,920,908
Ratio of 1933 to 1920 .	34.89%	25.57%	73.33%	67.29%

It will be noted that there were 34.89 per cent as many passenger one-mile units in 1933 as in 1920; 25.57 per cent as much revenue; and 67.29 per cent as many passenger-train miles. Both the volume of traffic and the revenue have fallen off much more than the number of passenger-train miles have been reduced, not because the railways have been unaware of the situation, nor because they have neglected the problem of reducing expenses as revenues declined, but because the nature of passenger business has made it impossible to reduce that service as rapidly as the patronage has failed.

When trains have ceased to earn their out-of-pocket cost of operation they have been discontinued or consolidated with other trains where practicable, and various methods have been used in reducing the cost of the remaining service. It has been possible in some instances to substitute bus service for passenger trains, the bus being the cheaper unit to operate where one bus will take the place of a train. Gas-electric cars have been used where they were of sufficient capacity to handle the business. The Chicago, Burlington and Quincy Railroad, for example, has 55 of these gas-electric cars, which in 1933 traveled 3,487,362 passenger train miles out of a total of 13,310,024, or 26.20 per cent. There remains a type of service which requires a larger train capacity than a single gas-electric car, but in many cases this service, even when operated with small steam locomotives, is so little used that it is unprofitable. However, the mail, express, and baggage which they carry generally render them necessary. Arbitrarily classifying such service as that which requires five cars or less, but more than one, it is found that approximately 25 per cent of the Burlington's passenger-train mileage is of this character.

An analysis of Burlington passenger statistics shows that the greatest loss in volume has been in the relatively short haul traffic. This is evidenced by the fact that in 1920, exclusive of suburban commutation travel, the Burlington carried 1,212,000,000 passengers one mile.



The Burlington Zephyr
On the Dotsero Cut-Off, June 16, 1934

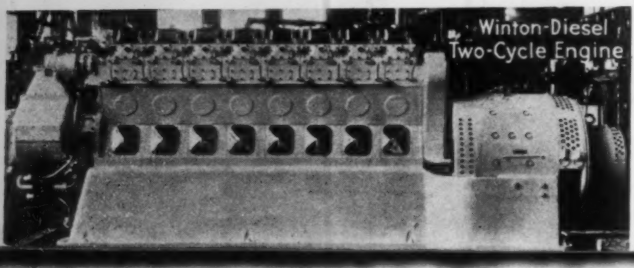
Of this traffic more than one-half was short haul. In 1933, the corresponding passenger-mile total was 304,152,000, of which about one-quarter was short haul. The average distance traveled in 1920 was 68.8 miles, and that in 1933 was 156.3 miles. The Burlington's experience is typical of American railroads as a whole. It is not that the people of the United States have been traveling less year by year—in fact, they have been doing more traveling than ever before—but that they have been using the highways rather than the railways. Most of this traveling has been done in private automobiles, contrary to a popular notion that buses are chiefly responsible for railroad losses in passenger traffic. Buses handle only a small amount of the total highway traffic.

ZEPHYR—A NEW RAILWAY UNIT

Since the greatest losses have been in local travel it is reasonable to seek ways and means of retrieving as much of that business as possible, and this was the thought which prompted the creation of the Burlington Zephyr. The aim was to give the very maximum of local passenger service of which a railway train is capable and at the minimum operating cost, in the belief that local passenger fares must remain low. Probably passenger fares have been cut so drastically in the West that the law of diminishing returns is in effect, and instead of enough traffic being recovered to offset the reduction, the decline in revenues has even been accelerated by these efforts.

It might well be said that the Zephyr was conceived as an oil-electric unit enlarged to make a complete train rather than a single car, although the plan from the beginning was to provide the maximum of travel comfort, which of itself is a distinct innovation in planning local service. It was also determined from the beginning to use a Diesel engine, if possible. Since the automobile industry, through its brilliant achievements in the design and manufacture of private automobiles, has been responsible for the losses of railway passenger traffic, it seemed logical to try adopting as many of its ideas as practicable. Therefore, in order to build into the Zephyr features that were foreign to conventional railway car construction well-known automobile manufacturers and noted architects were consulted. The streamlining embodied results of wind tunnel tests made by the Massachusetts Institute of Technology. The designs were made by the architect, Paul P. Cret of Philadelphia, with the architectural firm of Holabird and Root, of Chicago, acting as consultants. It was finally decided to have the train built by the Edward G. Budd Manufacturing Company, of Philadelphia. Perhaps it should be mentioned that, in spite of identical names, I can claim no relationship to the able president of that company.

It was decided to build the train of stainless steel and equip it with a two-cycle Winton-Diesel engine. As already announced by the Winton Corporation, "this is the first time in railroad history that an engine of this type has been used in this country in rail car operation." It is also the first completely streamlined steel train in this country. The train is of the articulated type—that is, a swivel joint connects the ends of two adjacent cars, which are carried on a single truck. The entire train of three cars is 197 ft long, weighs approximately 200,000 lb, and has only 16 wheels. The front truck carries approximately 100,000 lb, or half the weight of the entire train. The traction is applied to this truck by two d-c motors, one mounted on each axle. From engine room to observation lounge in the rear, there is provided in the following order 76 ft of mail, express, and baggage space, an electrically equipped buffet-grill compartment, and seats for 72 passengers (Fig. 1). The passenger compartments are air-conditioned throughout.



The Zephyr was planned for use between Kansas City, St. Joseph, Omaha, and Lincoln, where it will take the place of two locomotives and six cars, having a total weight of 1,618,000 lb, or eight times the weight of the Zephyr. Its daily round trip will total 502 miles. The route traversed is one of the finest local territories in the United States, but the people have ceased to ride on railroad trains to any great extent. With the idea of tempting them back to the rails, this latest innovation is offered for their approval. Whatever advantages light-weight, Diesel-powered, streamlined trains possess over other types will be demonstrated and their use extended accordingly.

For the conventional type of gasoline engine commonly used on railway motor cars, one or more carbureters are employed for mixing the fuel and air outside the cylinders, where the fuel atomization–time factor is not so important. In this type the ignition system can readily be adjusted to fire the combustible mixture to suit the higher speeds. In the other hand, the Diesel engine has neither carbureters nor electric ignition system. In it, air is compressed to approximately 500 lb per sq in. near the top of the stroke, raising the temperature to 1,000 F, which is more than sufficient to ignite the fuel oil injected into the combustion chamber at a predetermined moment.

The original Diesel engines, and practically all subsequent modifications until recent years, were of the slow-speed, stationary type. Formerly, a speed of 350 rpm was considered quite fast, whereas Diesel engines are now built to operate at speeds up to 1,800 rpm. The higher-speed Diesel involves higher gas pressures and greater forces of inertia, which in turn, require stronger parts to withstand the increased stresses and heavier bearing loads. In addition, the injection of fuel at higher speeds requires, for atomization of the fuel oil, pressures ranging from 2,000 to 18,000 lb per sq in. The timing and metering of the fuel also become matters of great precision. Such high speeds and pressures necessitated a complete redesign of the injection nozzles, valves, and associated mechanism to overcome the combustion lag, which would be very detrimental to efficient and satisfactory operation.

Low-priced, low-volatile fuel ranging from 22 to 36 deg Baumé gravity is burned in the Diesel engine. This fuel is considerably heavier than can be burned in a gasoline engine, and the difference in weight gives approximately 23 per cent more thermal units per gallon of fuel oil than for gasoline with the same number of British thermal units per pound. In addition to the fuel economy, the low-grade fuel oil burned in a Diesel engine reduces evaporation losses and fire hazard to a minimum. Gasoline will "flash" at a temperature of −50 F, but Diesel fuel will not do so until heated to a temperature of +150 F, even if it is exposed to an open flame.

This engine has 8 cylinders, of 8-in. bore by 10-in. stroke, and develops 660 bhp at 750 rpm. The auxiliary requirements, including air conditioning, lighting, cooking, radio, and air braking, require not more than 60 hp, leaving a minimum of 600 hp for propulsion. The train is driven by two 600-v traction motors, one mounted on each axle of the forward truck. Without the generator, the engine weighs 13,420 lb, or 20.8 lb per bhp. A two-cycle, slow-speed, marine-type Diesel engine, turning over at 104 rpm, weighs 216 lb per bhp. The four-cycle Diesel engines used in some oil-electric freight locomotives in this country operate at 550 rpm and weigh approximately 31 lb per bhp. Four-cycle gasoline engines, such as are generally used in rail motor cars which operate at 950 rpm, weigh 24.6 lb per bhp.

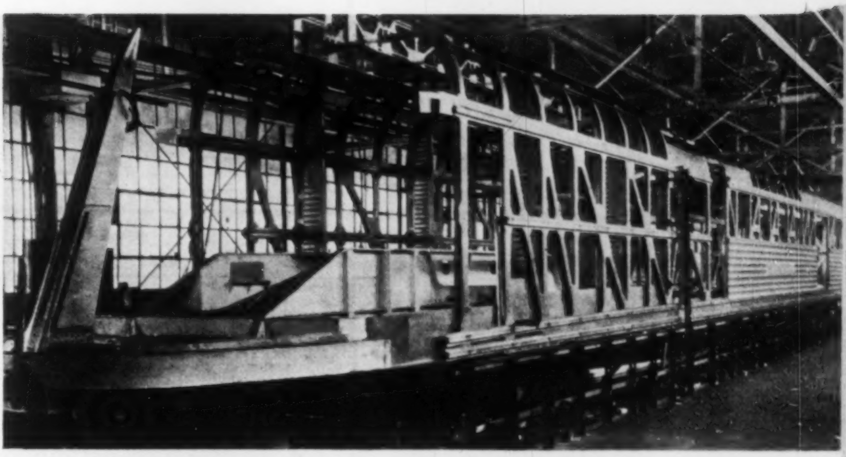
In the two-cycle Winton engine of the Zephyr, the efficiency is greatly increased through a scientific application of such devices as the Root-type scavenging air blower, which not only acts as a supercharger for augmenting the power output, but also provides the essential means for eliminating burnt gases from the cylinders. As altitude increases there is a power loss in gasoline- or distillate-burning engines not equipped with a supercharging device, which at an altitude of 5,000 ft amounts to about 15 per cent. The supercharger, which the Zephyr is the first rail car to use, prevents losses due to altitude.

The Zephyr's engine has a specially constructed water-cooled cylinder liner, near the bottom of which a series of air ports directly connect with the air-blower duct in which the air pressure is approximately 5 lb per sq in. when the engine speed is about 750 rpm. At the top of the cylinder liner is mounted a specially designed cylinder head with four valves, used for scavenging only.

HIGH-STRENGTH STAINLESS STEEL

For the Zephyr the light-weight construction afforded by stainless steel was considered admirable because the material is non-corrosive and insures permanency with a minimum of maintenance expense. Also, the polished surface affords the absolute minimum of skin friction. The "Shotweld" method of fabrication solved satisfactorily the most troublesome question, that of how to join members of stainless steel together so as to preserve the quality of non-corrosion and at the same time to utilize the full strength of the material. This process was developed at the Budd plant under the direction of Col. E. J. W. Ragsdale. Acknowledgment is gratefully made to him for much of the information about stainless steel here given.

Certain steels of the stainless group possess structural qualities even more remarkable than those responsible for the name. Chief among these is the so-called "18–8," steel alloy that contains 18 per cent chromium and 8 per cent nickel. This steel attains, without heat treatment, greater strength and a higher yield point than most tempered steels and at the same time has a toughness more in keeping with a softer material. In the annealed state it has a tensile strength almost twice that of mild steel and yet the workability of soft copper. In either condition, its resistance to corrosion is more like that of gold or platinum than of ordinary steel. Finally, to this unusual combination of strength and permanence is added a resistance to high temperatures such as is rare among metals. At temperatures where wood bursts into flame, where aluminum melts, and where iron bends of its own weight, "18–8" still retains enough of its strength to be as strong as is structural steel when cold. True, in so doing, it has lost some of its valuable properties, but the significant point is that it will not fail in a possible emergency.



Car Frame and Engine Bed—Front Car
"Shotwelded" Stainless Steel Construction

"SHOTWELDING" FOR FABRICATION

In making a "Shotweld," two or more sheets are held together by oppositely placed electrodes, and a current is passed between them. Resistance offered in the path of the current causes heating and, eventually, local fusion between the sheets. The total heat value is expressed as the product of the square of the current, the amount of resistance, and the length of time for which the current is applied. It is obvious that heat and, hence, the extent of fusion, is subject to accurate control. Further, it has been found that in this operation there are three functions having a distinct bearing on "18–8" stainless steel.

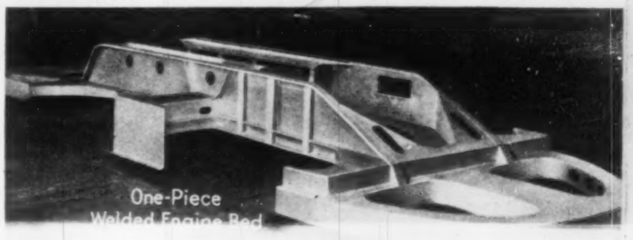
First, when "18–8" is heated above 2,000 F and quenched, it attains its softest state. Then it has an elongation of 65 per cent and, what is of further importance, a shearing strength of 75,000 lb per sq in. Both are approximately twice that of rivet steel. This is exactly what happens in "Shotwelding." The metal is locally fused and quenched in a chill mold of its own material.

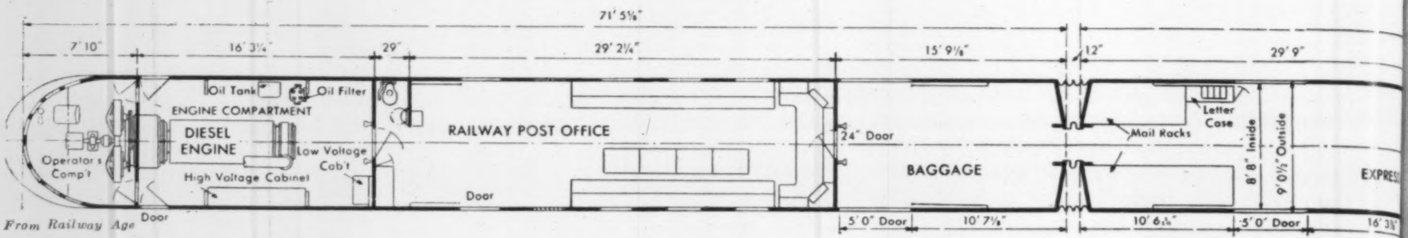
Second, the electrical resistance of "18–8" is from eight to ten times that of plain carbon steel; hence, the current required can be materially lessened. The clean surfaces of stainless steel add no complication.

Third, a peculiarity of "18–8" is that, if kept for any appreciable time in the critical temperature range of 900 to 1,500 F, certain carbides of chromium precipitate out and partially destroy valuable properties of the steel. The accent here should be on the word "time"; in the "Shotweld" process, great stress is laid on completion of the welding operation in a very small fraction of a second. The time of current dwell may range from 1/120 to 1/3 sec, depending on the thickness of the material to be welded.

Once these considerations became clear, the building of suitable apparatus was a relatively simple matter. More difficult was the development of a recorder to assure faithful performance, for it must be borne in mind that what one sees is not the weld but only the markings of the electrode contact. Actual fusion, in a good weld, is confined below the visible surface. Without some measure of heat value, reasonable doubt might exist as to whether or not sufficient fusion had taken place. The recorder supplies exactly this answer and gives a permanent record for every weld, good or bad.

Thus, the "Shotweld" process opens up a method of assembly superior to that offered by riveting. Each weld is not only twice as strong in shear as a rivet of like diameter, but the spacing can be made closer and there are no holes to upset calculations. Inciden-





From Railway Age

FIG. 1. FLOOR PLAN OF

tally, considerations of cost and speed decidedly favor welding. These make possible construction that otherwise could never be attempted.

ZEPHYR PERFORMANCE

The Zephyr was completed on April 8, 1934, and made its initial run from Philadelphia to Perkiomen Junction and return, 50 miles, on April 9. It has been exhibited in most of the principal cities of the United States, where more than a million people have passed through it. Burlington officials were desirous of having the benefit of certain information which could be obtained only by actual test runs, and accordingly it was planned to make a non-stop run at relatively high speed from Denver to Chicago. This was done on May 26. The time was recorded by Western Union. The Zephyr left the Denver Union Depot at 5:04 a.m., Mountain Time, and arrived at Chicago at 7:09 p.m., Central Time, a distance of 1,015.4 miles in 13 hr 5 min at an average speed of 77.6 mph. It continued without stopping to the Chicago lake front in Grant Park, making the non-stop run total 1,017.6 miles, and entered the transportation pageant, Wings of a Century, at the opening performance of the New Century of Progress Exposition. Some of the other records made on this run are shown in Table II.

TABLE II. SOME SPEED RECORDS MADE BY THE ZEPHYR ON MAY 26, 1934

FROM	TO	MILES	ELAPSED TIME	SPEED
Otis, Colo.	Chicago, Ill.	891	11 hr 16 min	79.0 mph
Otis, Colo.	Creston, Iowa	500	6 hr 18 min	79.1 mph
Otis, Colo.	Stratton, Nebr.	96.4	1 hr 4 min	90.0 mph
Otis, Colo.	Wray, Colo.	39.9	25 min 10 sec	95.1 mph
Otis, Colo.	Schramm, Colo.	19.1	11 min 35 sec	98.9 mph
Yuma, Colo. . . .	Schramm, Colo.	6.4	3 min 49 sec	100.1 mph

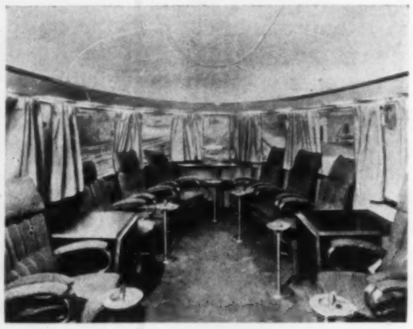
The maximum speed attained was 107 mph near Berwyn, Ill.

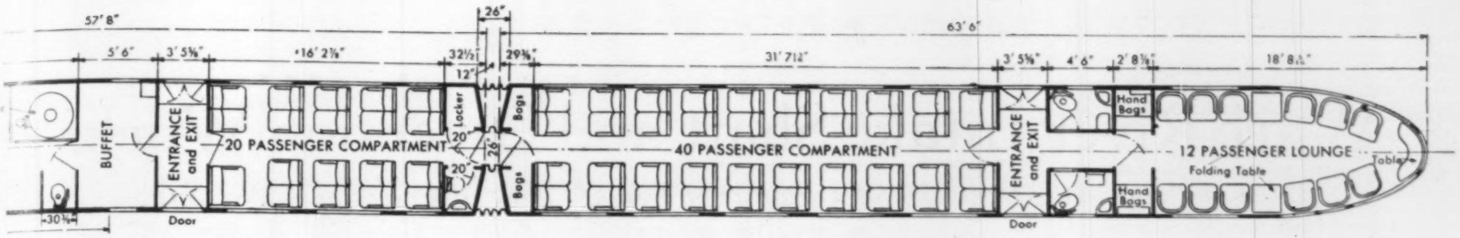
From Denver to Chicago the Burlington track is comparatively straight and the curvature is light. For example, 85 per cent of the line is tangent, and more than two-thirds of the curves are 2 deg or less. At six places the speed had to be reduced to 30 mph or less because of either temporary track over construction work or sharp curves, such as the approaches to the Missouri River bridge at Plattsmouth, Nebr., and to the Mississippi River bridge at Burlington, Iowa. The line from Ashland, Nebr., to Pacific Junction, Iowa, was used. This is a freight line, and on 32 miles of it the speed was restricted to 60 mph. Similar restriction was placed on the freight line $6^1/_2$ miles long, which was used at Albion, Iowa. The gradients are comparatively light, being 0.7 per cent maximum, except for $2^1/_2$ miles of 1.3 per cent gradients. There are few adverse grades from Denver to Chicago since Denver is at an elevation of 5,170 ft above the sea while Chicago is at an elevation of only 538 ft.

On June 16, at Bond, Colo., the Zephyr participated in the ceremonies at the opening of the Dotsero Cut-off of the Denver and Rio Grande Western Railroad Company and proceeded thence to California, being the first train to reach California over that line. On July 3, it ran from mile post 416 to mile post 411 near Otis, Colo., a distance of 5 miles, at an average of 107.62 mph; also from mile post 402 to mile post 397 near Yuma, Colo., a distance of 5 miles, at an average of 106.98 mph., with a maximum of 108.38 mph between mile posts 412 and 411. Several records for single miles, which were made with stop watches, have shown speeds from 112 to 116 mph. However, they are unofficial, and the records here given were recorded and confirmed by Western Union. Important data of two general types were gained from these and other high-speed runs: first, as regards track, and second, as regards train performance, that is, power plant and streamlining.

On the non-stop run from Denver to Chicago volunteer forces cooperated to keep the spectators lining the right of way, estimated to have been upwards of 1,500,000, far enough away from the track to avoid delay and accident. Neighboring railroad lines were advised of the Zephyr's schedule and considerately avoided interference at railroad crossings, while Government officials obligingly had navigable streams patrolled to keep any craft from approaching drawbridges when their opening might have necessitated stopping the Zephyr.

Either the Engineer of Maintenance or the Assistant Chief Engineer rode in the front end of the train the entire distance with a track chart before him. On this had been marked the permissible speed on all curves, there being no speed restrictions on tangent track. He checked with the motorman to see that the speed was proper on curves and made critical notes commenting on the riding quality of the track. The Chief Engineer rode in the rear of the observation-lounge with another track chart on which he recorded his observations. The permissible speed was based on a compromise between those considered comfortable and those considered safe according to the recommendations of the American Railway Engineering Association regarding super-elevation. The entire trip was made without any compromise with safety, and only on one curve was there slight discomfort. That was a 2-deg curve on which the super-elevation was reported as 4 in. and the permissible speed was marked at 80 mph. The actual speed was 82 mph, and a careful check revealed that the super-elevation near one end of the curve was $3^1/_2$ in., with a maximum of $4^1/_4$ in., the change from $3^1/_2$ to $4^1/_4$ in. accounting for a lurch which was felt on the train. To provide for regular operation of the Zephyr at upwards of 80 mph—for which a table of super-elevations taking into account its lower center of gravity has been prepared—would require some increase in super-elevation of the outer rail and longer spirals, with the attainment of full super-elevation spread out over a greater distance. For example,


OBSERVATION LOUNGE IN THE ZEPHYR

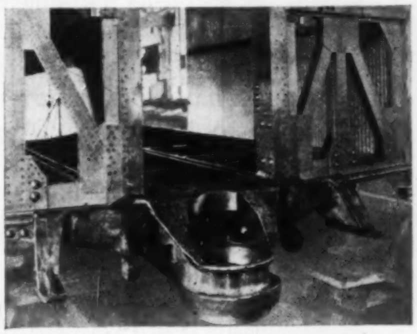


THE BURLINGTON ZEPHYR

a 2-deg curve for a speed of 80 mph should have 5 in. of super-elevation and spirals about 400 ft long. The super-elevation should begin on the level track 120 ft beyond the points of the spirals and should increase uniformly from zero to full super-elevation at the beginning of the circular curve. At 80 mph this would mean that the train would acquire superelevation at a rate of $1^1/_8$ in. per sec in entering the curve and lose it at a corresponding rate while leaving the curve.

The performance of the Zephyr showed conclusively that the low center of gravity and articulated construction give more safety and much greater comfort at high speeds on curves than is possible with a train made up of conventional cars. A measure of current input for propulsion, and for other purposes, indicated that a minimum of 600 hp was available for propulsion at all times. At the end of the 1,018-mile run the engine and motors were in perfect condition and ready to continue indefinitely without stopping.

During this trip an incident happened which was not important but was made to seem so by a sensational and erroneous report given rather wide circulation. On the afternoon of May 25, when receiving its final inspection at the Denver shops before the non-stop run to Chicago, the outer race of the roller bearing at the commutator end of the armature shaft of the No. 2 traction motor was found to be cracked across. This was probably due to a hardening strain in the roller race. The cracked race gradually destroyed the rollers and eventually let the armature fall low enough so that the commutator was riding on the lower brush holders—a brass casting. A new bearing—secured from the Union Pacific—was fitted into place. The repairs delayed the departure for Chicago 1 hr and 5 min beyond the time scheduled.



ARTICULATED CONNECTION AT END OF CAR

On its recent trip to California over the Denver and Salt Lake Railway the Zephyr made the ascent from Denver to Moffat Tunnel, a rise of 4,072 ft on practically a continuous 2 per cent gradient, at the maximum speeds permitted by the curvature. It demonstrated its ability to make approximately 42 mph on that grade. Returning from California by way of the Denver and Rio Grande Western Railroad, it ascended the Tennessee Pass, reaching an elevation of 10,239 ft on a 3 per cent grade at a sustained speed of 25 mph.

On the night of May 1 near Essex, Ontario, on the Michigan Central Railroad the Zephyr struck a truck which had stalled on the track and was partly tipped over, two wheels on one side being off the crossing planks and down on the ties. The speed of the Zephyr was 60 mph when the truck loomed up and 40 mph when the collision took place. It stopped about 400 ft beyond the accident. No damage was done to the Zephyr but the truck was completely demolished. Fortunately, no one was injured because the truck had been stalled long enough for the driver to get off.

Up to July 6 the Zephyr had run 15,000 miles. It can be said with entire truth and candor that no inherent defect has developed either in the design and construction of the train or in the power plant. On the night of July 3 one of the plugs in the core hole in cylinder No. 8 gave way. Design of this detail, which was of a rubber composition expanded between two discs, has been changed to provide for a screw plug. The only other incident that has happened to the machinery aside from the cracking of the outer race of the armature bearing on May 25 was the loosening of a gear on the same motor on June 13. This was due to the fact that the gear was not securely attached when a new armature was installed in place of the one involved by the defective bearing on May 25.

To state that the Zephyr is an experimental train does not mean that its performance is in doubt. The various parts are assembled in a new relationship to each other and the result is unique, especially considering the streamlined exterior and the large composite structure of "18–8" stainless steel. But each of these parts has been carefully designed and tested even to such new features as the 2-cycle Diesel engine, the automatic brake control, and the thin crumpled aluminum sheets for insulation material. There is no longer any doubt about the practicability of a train of the Zephyr's capacity or even of larger units, for high-speed, low-cost operation. The experiment is with the travel habits of the people, to determine whether anything, or what things can be provided in a railway train to induce people to travel locally by rail again to any great extent.

The design and construction of such a train as the Zephyr was possible for the first time in 1933. It could not have been built earlier because a light-weight two-cycle Diesel engine of the kind required was not available, and only a short time before had the method of fabricating stainless steel by "Shotwelding" been perfected. The use of stainless steel and welded alloys, such as "Cromansil," are responsible for the light weight of the train, and of course both are essential features. They in turn are the result of progress in metallurgy. It was only very recently that engineers had placed in their hands new materials of so much greater unit strength than ordinary structural steel as to upset their previous ideas of the relationship of mass and weight to the strength of materials.

It seems reasonable to presume that since these developments are so recent they will be carried much further and that still more amazing results may be attained. What changes the application of the materials, principles, and methods of design and construction used in the Zephyr may make in the construction of equipment, motive power, and fixed structures can only be conjectured. There has been a growing belief that railway freight equipment is too heavy and that the ratio between the weight of the typical freight car and its load could be greatly improved. The Burlington Zephyr may herald definite progress in this direction.

Ware River Intake Shaft and Diversion Works

Design and Automatic Operation of a Feature of Boston's New Metropolitan Water Supply

By Karl R. Kennison

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In the June issue Frank E. Winsor, M. Am. Soc. C.E., described the principal features of the Quabbin Aqueduct water supply system of Metropolitan Boston, now under construction. It is to consist of a 25-mile tunnel being driven through the high ground west of the present Wachusett Reservoir to connect with the Quabbin Reservoir, when completed. In this article Mr. Kennison explains the purposes and the construction of the shaft located about midway between the Wachusett and the Quabbin Reservoir and the operation of the automatic devices that control diversion. Through this shaft the flood flows of the Ware River are dropped into the new tunnel, which is 260 ft below the surface at this point. When the tunnel is finished, 3,100 cu ft per sec can be diverted, part to each of the reservoirs. The diversion equipment, now completed, may be operated entirely automatically, so that the siphons go into action whenever the river flows at a greater rate than 85 mgd. A unique feature of the shaft is its cast-iron lining with helical vanes, which absorbs most of the energy of the falling water and prevents damage to the shaft and tunnel. The present state of the work permits diversion to the Wachusett Reservoir only. Amounts approaching 1,000 cu ft per sec have been successfully diverted to this reservoir.

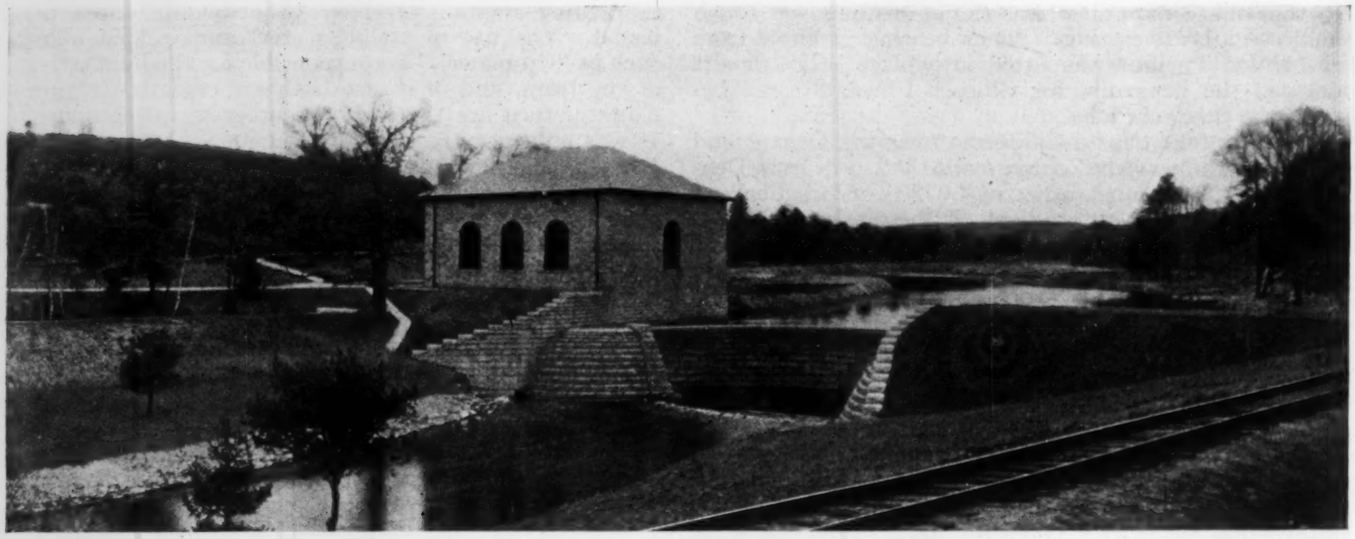
At the top of Shaft No. 8 of the Quabbin Aqueduct tunnel is located the Ware River intake. This shaft is about 260 ft deep. It was used as a construction shaft during the excavation of the tunnel and was afterward lined with cast-iron plates and used as an intake shaft to divert into the tunnel the flood flows of the Ware River.

By Massachusetts legislation the diversion is limited to flows in excess of 85 mgd; that is, on any calendar day when the flow is at a rate lower than this, no water can be diverted. On days when water is diverted it now flows to the east through the tunnel for 13.2 miles to Wachusett Reservoir. The tunnel is also being extended to the west 11.4 miles to the Swift River, where the Quabbin Reservoir will be constructed to store about 415 billion gallons (1,274,000 acre-ft), or six times as much as the Wachusett Reservoir. In future the storage of these Ware River floods will be principally in the new Quabbin Reservoir, although all or part of the flow can be diverted easterly to the Wachusett Reservoir by proper manipulation of the tunnel gates at that end. The diversion dam is only about 17 ft high. The intake works at one end of this structure are designed to skim off the flood flows at rates varying from zero to a total which, when the tunnel is completed to Quabbin Reservoir, will be approximately 2,000 mgd (3,100 cu ft per sec) from a drainage area of 98 sq miles. Not more than the specified 85 mgd (131 cu ft per sec) will be wasted down the stream.

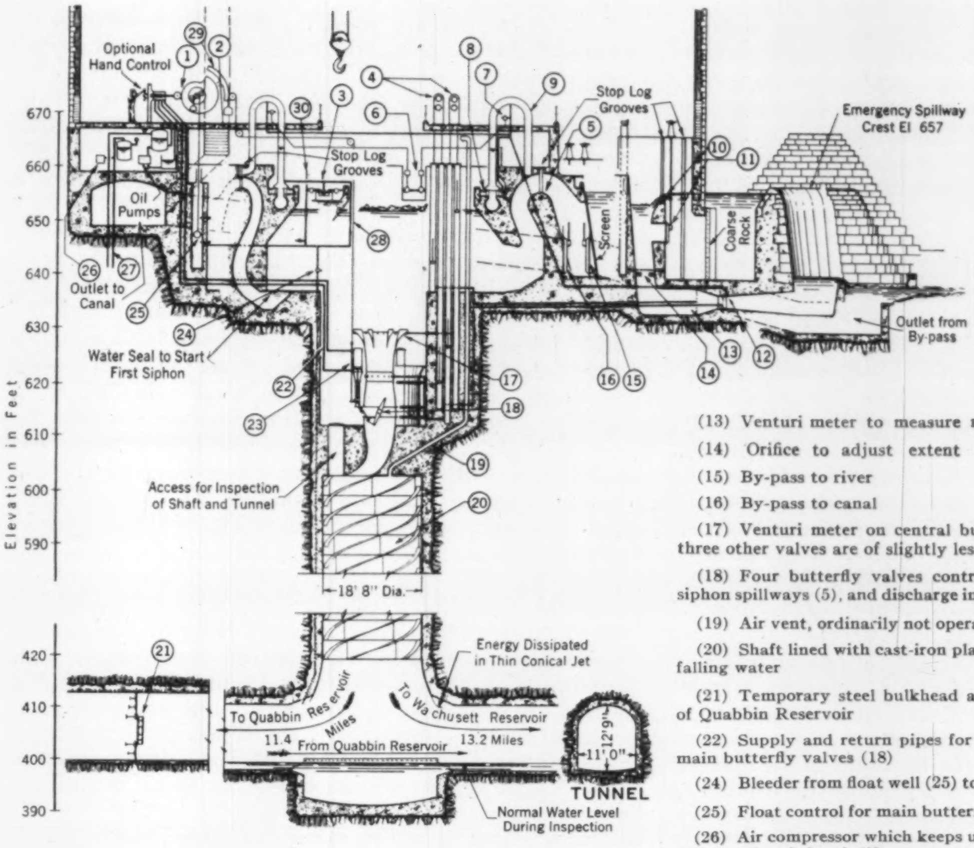
SIPHON SPILLWAYS AND BUTTERFLY VALVES

Directly over the shaft is the intake building, which is 78 by 65 ft. Along two of the sides, in the substructure, are intakes to nine siphon spillways varying in capacity from 60 to 550 cu ft per sec under a 9-ft head. The six larger siphons have throats 108 by 39 in. in cross section. These spillways discharge into a central well or valve pit in which the water level, and thereby the head on the siphons, is controlled so that the total available flood flow is automatically skimmed from the small diversion reservoir and taken into the shaft.

In the bottom of the central well, and directly over



Ware River Intake Works Completed



(1) Four cams, float-controlled, which actuate oil sleeve valves to open and close main butterfly valves (18)

(2) Louver to maintain atmospheric pressure inside building under fluctuating water level

(3) Compensating weir for automatic regulation of flow to float well (25) for control of main butterfly valves (18)

(4) Float-operated meter registers to record rate of diversion through (17) and release through (13)

(5) Nine siphon spillways with a crest at elevation 656—six of 360-mgd capacity under maximum 9-ft head, and three smaller. Order of priming determined by sensitive float switches (6) operating solenoid valves (7) on vacuum connections. Order of breaking of siphons determined by surface contact (8) of siphon break pipes (9)

(10) Pond level at diversion limit for release of 85 mgd fixed at elevation 656 by adjustable weir crest (10) or sluice gate (11)

(12) Vacuum pipe connected with Venturi meter (13) for rapid priming of siphons (5)

(13) Venturi meter to measure release to river

(14) Orifice to adjust extent of backwater on weir crest (10)

(15) By-pass to river

(16) By-pass to canal

(17) Venturi meter on central butterfly valve of 620-mgd capacity. The three other valves are of slightly less capacity and are unmetered

(18) Four butterfly valves controlling water level in valve pit, head on siphon spillways (5), and discharge into shaft

(19) Air vent, ordinarily not operative on account of vacuum in shaft

(20) Shaft lined with cast-iron plates with helical vanes to absorb energy of falling water

(21) Temporary steel bulkhead and flap gates in use pending completion of Quabbin Reservoir

(22) Supply and return pipes for oil pressure cylinders (23) operating the main butterfly valves (18)

(24) Bleeder from float well (25) to bottom of shaft

(25) Float control for main butterfly valves (18)

(26) Air compressor which keeps up required pressure in water and oil tanks and supplies air for air-lift water supply (27)

(28) Steel cable connecting each butterfly valve to compensating arm (29) on oil valve control

(29) Compensating arms to prevent hunting in opening and closing of main butterfly valves (18)

(30) Air vent

FIG. 1. DIAGRAM OF WARE RIVER INTAKE WORKS

the shaft, are four butterfly valves. These are set with vertical axes and their discs are held automatically at just the right angle to pass the available flow in excess of 85 mgd in the river. They also hold the water in the central well or valve pit at just the right level so that the siphons will pull this quantity from the river.

These four valves discharge through the shaft cap and tangentially against the side walls of the shaft at an angle of about 30 deg with the horizontal. The cast-iron plates with which the shaft is lined throughout the 184 ft between the valves and the top of the tunnel, were cast with projecting ribs, which form helical vanes like a six-pitch screw thread. A photograph shows the top of the shaft and the valve pit during construction. The vanes ensure the maintenance of the rotary motion of the falling water throughout the entire drop to the tunnel. Thus the water is pressed firmly by centrifugal force against the shaft wall and is given no opportunity to impinge directly against any surface. As a result, the energy is dissipated in a thin conical jet at the bottom. Before final designs were made, the action was studied with the aid of a model in the Alden Hydraulic Laboratory of the Worcester Polytechnic Institute.

In the normal future operation of the tunnel, the hydraulic gradient will stand about 120 ft above the tunnel invert. However, until the Quabbin Reservoir is completed and tunnel control gates are installed, and even after such installation whenever these gates are open to permit free discharge easterly to the Wachusett Reservoir, water diverted at a rate as high as 400 cu ft per sec will drop the entire distance to the tunnel into a pool only about 15 ft deep. As the rate increases to about 450 cu ft per sec, the bottom of the shaft is sealed, creating a vacuum in the shaft equivalent to about a 25-ft head of water, which increases the depth of the water cushion to that extent. Further increases in the rate of diversion lift the hydraulic gradient proportionately until the capacity of the works is reached, when there is no residual head to lose in the drop into the shaft.

One important function of the water in the central well or valve pit, in addition to its control of the head on the siphons, is its action as a seal to prevent the entrance of air into the tunnel. It is on this account that the high vacuum previously referred to is produced in the shaft when its bottom also is sealed. This vacuum has several important effects: it increases the discharge capacity of the butterfly valves; it increases the vertical loading for which the shaft cap had to be designed; and it also makes necessary an ample depth of water above the entrance to the valves to prevent the formation of a vortex, which would connect the vacuum to the superstructure of the building. The windows of the building are equipped to a limited extent with swinging louvers, and a large set of louvers especially designed to swing freely in both directions is provided to maintain atmospheric pressure inside the building under the normal fluctuation of the water level in the valve pit, and also to function in case of a vortex through one of the valves, or in any other emergency.

A diagram of the automatic devices which control the rate of diversion is presented in Fig. 1. The relative positions and the scale of various parts have been distorted to a limited extent, as necessary to make the diagram clear.

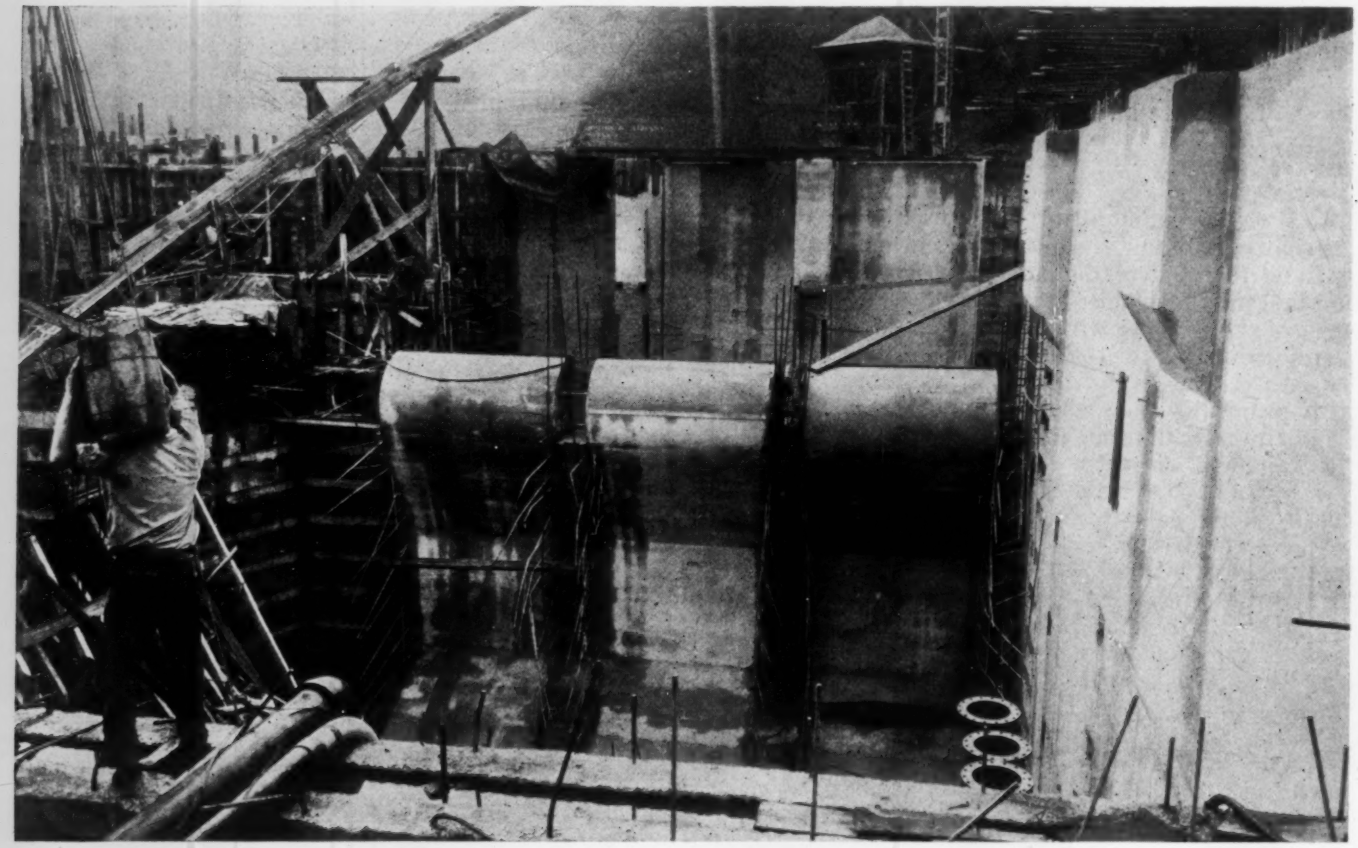
The required release, up to 85 mgd, flows over two weirs in the building, each having a crest length of 9 ft. One weir crest is adjustable, so that as the natural flow increases and reaches this limit, the pond stands at elevation 656 above mean low tide, or one foot below the crest of the diversion dam. This foot of leeway is allowed in the operation of the works, there being no flow over the spillway down the river so long as the pond is held at its proper low level. As a matter of fact, the permitted fluctuation in the pond level under normal operation is less than 0.1 ft, the entire foot of leeway being provided to take care of any abnormal surges. The reason for keeping the pond as closely as possible at the fixed level of elevation 656 is to maintain a constant flow over the weir. Operating experience to date indicates that the fluctuations of the flow over the weir due to slight changes in pond level do not increase the average quantity released sufficiently to be of any consequence. However, if desired, the flow can be held more closely to the required rate of 85 mgd by using a sluice gate provided for the purpose as an orifice under a head of about 5 ft instead of the weir, but to date it has not been considered necessary to use the sluice.

The pond level extends into the building to the siphon regulating trough, built around two sides of the valve pit. The connecting pipe extends under and along the entire length of the trough and has openings to the trough under each break pipe. The pond level is also connected to a well containing float switches, which are instrumental in the rapid priming of the siphons, as explained later. The siphon regulating trough is used principally in the automatic breaking of the siphons at just the right time in order to prevent the diversion of any part of the 85 mgd specified to be released. This trough is divided by partitions between the break pipes, which fill its entire cross-sectional area and prevent any surface waves from traveling along the trough from one break pipe to another.

Each break pipe is flared at the bottom to increase the circumferential area under the lip, where the air is sucked in to break the siphon. The surplus water that falls down the break pipe when the siphon is broken is guided and continued down through the bottom of the trough directly to the large pipe connection to the pond. The adjustable lips on the siphon break pipes are set at elevations which vary only a few thousandths of a foot from each other, and the siphons are broken in succession, as required, with practically no disturbance of the controlling water level in the trough. The siphon break pipes project above the main floor of the diversion building.

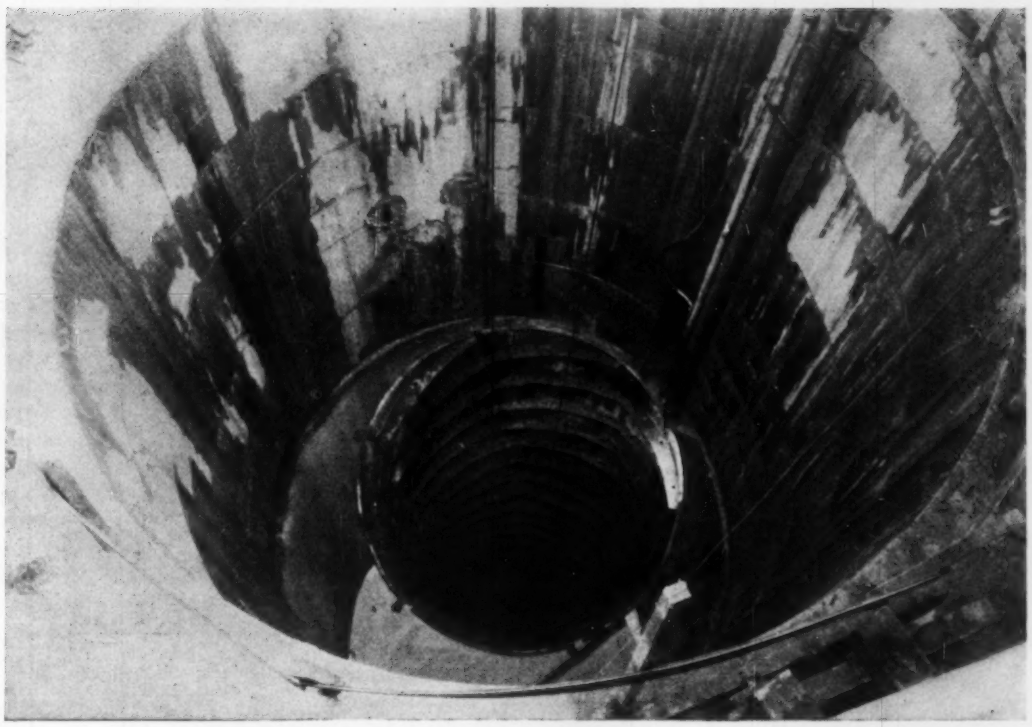
RAPID PRIMING OF SIPHONS

A Venturi meter, which records the rate of discharge allowed to continue in the river up to 85 mgd, is designed with a throat diameter smaller than would be dictated by ordinary requirements. The vacuum system of piping running throughout the basement is connected to a specially designed header just downstream from the throat. The result is that whenever there is sufficient flow to permit diversion, the system is under a considerable vacuum, which is used to assist rapid priming of the siphons. This is accomplished by solenoid valves, one on a connection from the vacuum pipe system to the high point of each siphon break pipe.



SIPHON SPILLWAY UNDER CONSTRUCTION
The Throats of the Three Siphons Shown Are 108 by 39 In.

These solenoid valves are actuated by sensitive float switches set to prime the siphons in a predetermined order, with only a few thousandths of a foot difference in the pond level between the priming of any two siphons or group of siphons. These float switches and solenoid valves are a refinement in the interest of holding the river flow down to the specified 85 mgd and are not a necessity, since the siphons would be sure to prime themselves under much less than the leeway of one foot of head allowed in the operation of the works. In other words, a failure of the electric current cannot stop the priming of the siphons.


Shaft Lining of Cast Iron with Helical Vanes

Four butterfly valves discharge through the shaft cap and maintain the water seal in the valve pit. They are operated by oil-pressure cylinders directly connected to the valve castings and are kept from hunting or racing by a flexible steel-cable connection to the control panel in lieu of the piston tail rod ordinarily used for this purpose. Thus the only required mechanical connection to these valves in the deep pit is a flexible cable, which can adjust itself to the swirl of the water without causing undue strains. Also, on account of the nature of these connections, the control panel can be conveniently located in one corner without obstructing the main floor. It distributes the oil to the opening or closing pipe of each valve, as required, by means of four small oil sleeve valves, one for each of the main butterfly valves.

These sleeve valves can be operated manually or left on automatic control, at the will of the operator. When on manual control any or all of the main butterfly valves can be held open at any fixed, predetermined angle. This could be done, for example, to reduce the circulation of oil to a minimum in case a failure of the electricity required the operation of the emergency hand oil pump. A photograph shows the small, compact operating panel and the meter registers.

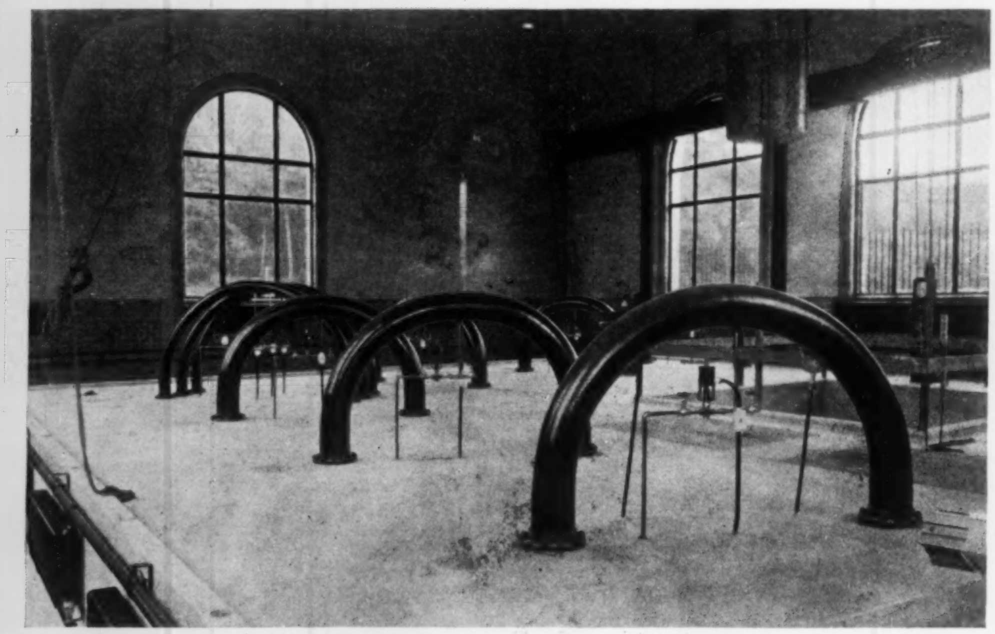
AUTOMATIC CONTROL DEVICES

For the automatic control, special devices are necessary because the prime moving force is nothing more than the slight permissible variation in the level of the diversion pond. First, it was obviously necessary, as in the case of any such mechanism for throttling or partially closing a valve, to provide a compensating device to prevent hunting. In this case the cable connection to the cross head of the oil cylinder permitted a simple, compact design, which is illustrated in Fig. 1. The cam that actuates each oil sleeve valve is driven through planetary gears attached to the compensating arm.

Second, the main butterfly valves are opened in a fixed and predetermined succession. It was desired to obtain a meter record of the rate of diversion under all conditions except those of very high floods, and it was economical to construct a Venturi meter in the bell mouth approach to one valve only and to arrange the sequence of opening so that the other three valves would remain closed until the first was wide open, and so that the other three would then be opened in predetermined succession, one after the other. To control the valves as thus outlined, a large float well is provided in one corner of, and not directly connected with the valve pit. The relatively small fluctuations in the pond level produce wide fluctuations in this float well, the total distance of float travel being about 40 in. This float actuates the main cam shaft on the control panel, and the cams which operate the oil sleeve valves can be set at any desired angle on this shaft so as to secure the desired sequence in the opening of the main butterfly valves.

Third, smoothness of operation is increased by a device actuated by the rate of rise and fall of the diversion pond rather than by its actual level. Instead of holding the pond as closely as possible to one fixed level and regulating the flow into the float well accordingly, the flow to or from this well is regulated by the rapidity of rise and fall of the diversion pond within the small range of 0.024 ft. In other words, if the pond is at elevation 656, or 0.024 ft higher, or anywhere within this range, the regulating valves are not moved. They are moved only by the slight tendency of the pond to rise or fall, and then in proportion to the rate of such rise or fall. This is accomplished by a small compensating weir in the shape of a circular bowl supported by floats in the siphon regulating trough, as shown in Fig. 1. This weir is floated between stops that permit the slight vertical travel of 0.024 ft, so that its crest tends always to remain slightly beneath the water surface. The resulting small flow of water into the bowl is sucked up as soon as diversion is started by the priming of the first siphon and is conducted through a system of piping to a free vent down the shaft into the tunnel.

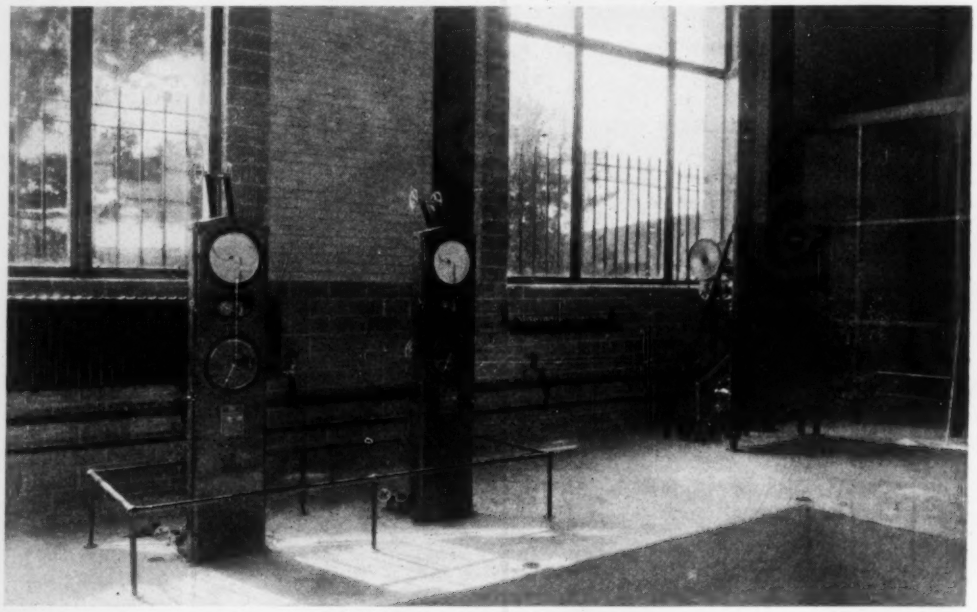
The float well is connected to this system of piping so that its water level always corresponds to the hy-



SIPHON BREAK PIPES ON INTAKE BUILDING

draulic gradient in the piping. This, in turn, on account of the free vent into the tunnel, is entirely independent of, and unaffected by, any possible condition of backwater in or about the intake works. It depends solely on the rate at which water is sucked out of the bowl of the compensating weir. This rate is constant so long as the compensating weir floats freely in the siphon-regulating trough. When the pond rises so that the compensating weir strikes the stop at the upper limit of its 0.024 ft of travel, the flow into the bowl is immediately and rapidly increased; the level in the float well rises; and the main butterfly valves are opened. Similarly, when the pond level drops so that the compensating weir strikes its lower stop, the flow into the bowl is cut off and the system of piping continues to drain into the tunnel, thereby pulling down the level in the float well and shutting the main butterfly valves.

Furthermore, the bowl of the compensating weir is set tightly into an outer bowl like a piston with a loose, sliding fit. Since the entire device is submerged in, and filled with water, it acts like a dashpot so that the compensating weir, instead of floating freely, is sluggish and always trails the rise and fall of the pond to an extent proportional to the rapidity of such rise and fall. As a result, the compensating weir anticipates, so to speak, the necessity for the opening or closing of the main valves and puts an additional damper on any tendency to hunt or race. Tiny adjustable orifices within the inner and outer bowls make it possible to adjust the sluggishness of the motion, or the extent to which the compensating weir trails the rise and fall of the pond, in order to secure the greatest freedom from surges in actual operation.



VENTURI METER REGISTERS AND VALVE CONTROL PANEL

The section of the tunnel to Wachusett Reservoir and the substructure of the intake works were completed in time to divert the spring flood flows of 1931. The superstructure of the building had only just been started, but the heating plant had been installed in the basement and small temporary wooden housings had been built around the control panel, the siphon break pipes and solenoid valves, the switchboard, and other apparatus above the main floor of the building so that the works could be put in operation.

Water was first available on March 21, 1931. The works were successfully operated throughout the entire period of high flows, and approximately 13 billion gallons were diverted from March 21 to June 14 of that year, inclusive. The maximum rate of diversion this first season was 3,685,700,000 gal in the week commencing March 29 and, in a single day, an average rate of about 30 tons (960 cu ft) per sec. This high rate of diversion, coming so soon after the plant was put in operation, constituted a severe test of the intake works.

At first, operation was largely by manual control in three shifts, at least one operator being on hand at all times. Commencing April 19, the works were successfully placed in completely automatic operation, with an operator on hand only during the working hours of the day. The diversion of the allotted flood flows continued without attendance nights, Sundays, and holidays, entirely automatically except for several occasions during the daytime when manual control was used for special purposes.

Bouquet Canyon Reservoir and Dams

Large Earth-Fill Structure Provides Needed Storage for Domestic Supply of Los Angeles

By H. A. VAN NORMAN

MEMBER AMERICAN SOCIETY OF CIVIL ENGINEERS
CHIEF ENGINEER AND GENERAL MANAGER, BUREAU OF WATER WORKS AND SUPPLY, DEPARTMENT OF WATER AND POWER, CITY OF LOS ANGELES, CALIF.

TO provide additional terminal storage for the Owens River Aqueduct below the San Andreas Fault, nearly three million cubic yards of earth and rock have been placed across the Bouquet Canyon 50 miles north of Los Angeles to create a reservoir with a capacity of 38,800 acre-ft. Carefully selected, well-graded material was excavated from a pit within the reservoir area and dumped, spread, and compacted with rollers of the sheep's-foot type to form the impervious part of the barrier, which rises 221 ft above bedrock. The downstream pervious section was built of material stripped from bedrock in the dam prism and adjacent to the abutments. On the upstream face a reinforced concrete slab was placed and joined to a concrete cut-off wall along the upstream toe, which was carefully grouted as an additional precaution against leakage. Other structures of interest described by Mr. Van Norman are the spillway shaft, the gate-control tower for the inlet-outlet pipe line, and the provisions for releasing the normal run-off from the Bouquet Canyon drainage area to downstream users. After a two-year construction period, water brought to the reservoir from the Owens River Aqueduct was first placed in storage on March 28, 1934.

IN order to serve the growing population of Los Angeles with a more adequate and more dependable supply of water, the Bureau of Water Works and Supply, Department of Water and Power of the City, has constructed a new reservoir in Bouquet Canyon. This reservoir, about four miles to the east of the Los Angeles–Owens River Aqueduct and 50 miles north of the city, will create additional stand-by storage south of the San Andreas Fault. Thus it will assure continuous service in both water and power in the event of unavoidable interruption of the aqueduct supply from the north.

Bouquet Canyon is a deep and narrow gorge about 15 miles long, extending in a northeasterly direction from its confluence with the Santa Clara River Valley about two miles northwest of Saugus, Calif. Some of the peaks in the vicinity rise to an elevation of 5,000 ft above sea level, but the average elevation of the high ridges is about 4,000 ft. The upper part of the canyon cuts into

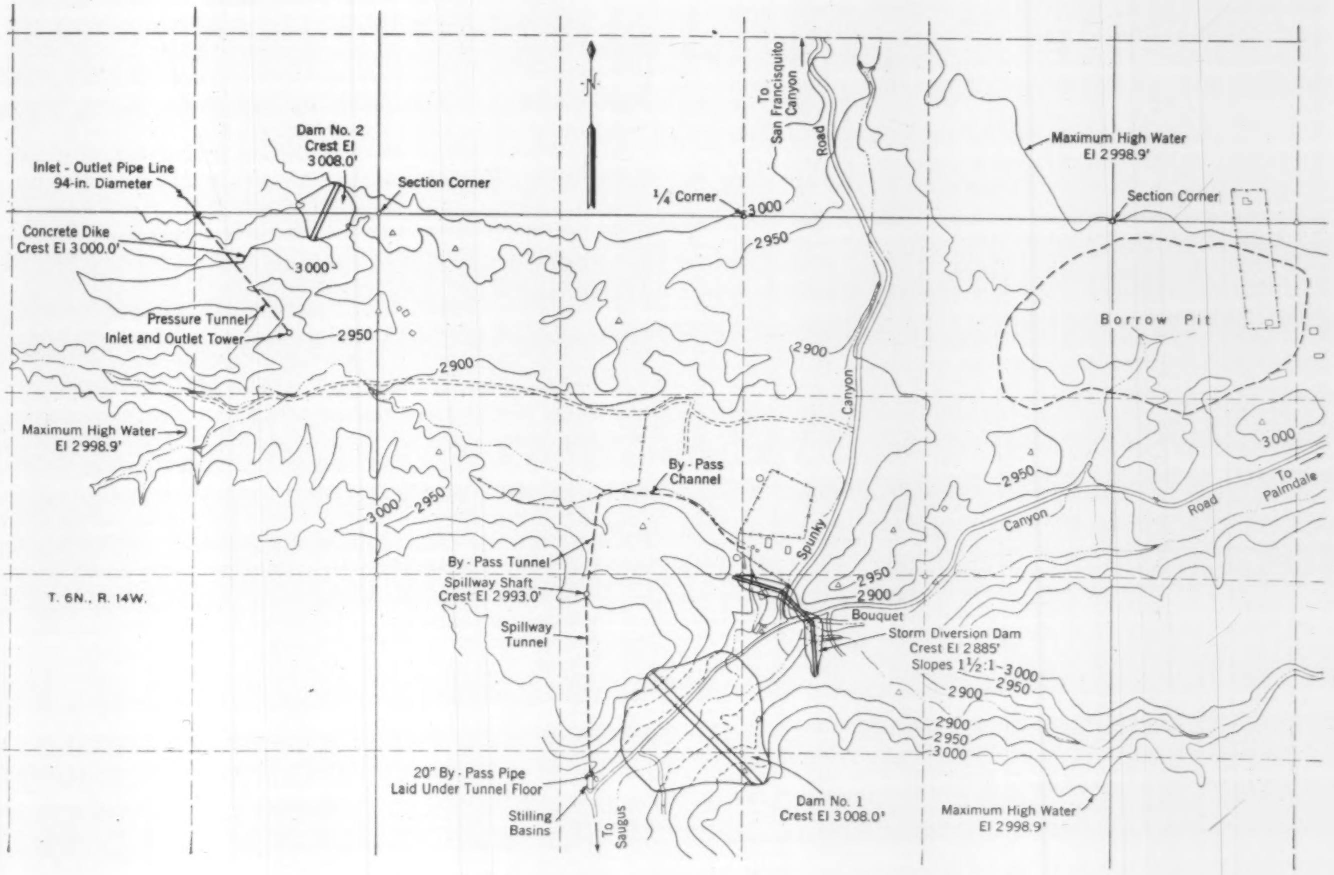
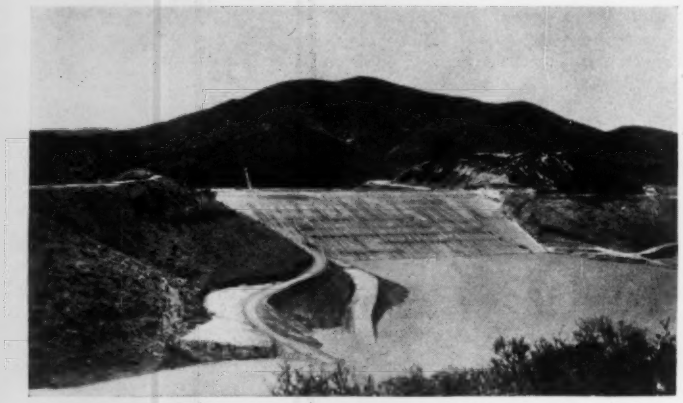

N
To San Francisquito Canyon
Road
Dam No. 2
Crest El 3008.0'
Inlet - Outlet Pipe Line
94-in. Diameter
Section Corner
¼ Corner
3000
2950
Maximum High Water
El 2998.9'
Section Corner
Concrete Dike
Crest El 3000.0'
3000
Pressure Tunnel
Inlet and Outlet Tower
2950
Borrow Pit
2900
2900
Canyon
Maximum High Water
El 2998.9'
3000
By - Pass
Channel
2950
3000
2950
Canyon
Road
To Palmdale
Spunky
By - Pass Tunnel
Spillway Shaft
Crest El 2993.0'
2950
2900
Bouquet
T. 6N., R. 14W.
Spillway
Tunnel
Storm Diversion Dam
Crest El 2885'
Slopes 1½:1
3000
2950
2900
2900
2950
3000
20" By - Pass Pipe
Laid Under Tunnel Floor
Stilling
Basins
To Saugus
Dam No. 1
Crest El 3008.0'
Maximum High Water
El 2998.9'

FIG. 1. BOUQUET CANYON RESERVOIR AND APPURTENANT STRUCTURES

schists, shales, and sandstones, although the core of the mountain mass to the north is granitic. Where the gradient is not steep, the bottom of the canyon has been filled by alluvial material consisting of sand, gravel, and fine earth.



BOUQUET CANYON DAM NO. 1 AS COMPLETED IN MARCH 1934

The Los Angeles–Owens River Aqueduct approaches the city by way of the San Francisquito Canyon, which is the next major canyon to the west of Bouquet. These two canyons are separated in their upper reaches by a distance of between three and four miles. The intervening ridges and hills are characterized by steep slopes and are dissected by steep-walled and somewhat tortuous canyons.

Situated in the upper part of Bouquet Canyon and nearly due east of Municipal Power Plant No. 1, the newly constructed reservoir is supplied with water from the Fairmont Reservoir of the Los Angeles–Owens River Aqueduct. Water from that reservoir flows through the Elizabeth Lake Tunnel and into the penstocks of Power Plant No. 1. The newly completed inlet-outlet pipe line, which was described in the June issue, connects these penstocks with a tunnel through the ridge at the western side of the Bouquet Canyon Reservoir. Through it water is either stored in, or withdrawn from the reservoir. The major reservoir structures consist of two earth and rock embankments, a concrete dike, and a vertical shaft spillway, as described here.

MAIN DAM OF EARTH AND ROCK FILL

Across the main canyon the main dam, Dam No. 1, has been constructed to a height of 185 ft above the stream bed of Bouquet Creek, or about 221 ft above bedrock. It closes the southern end of the reservoir at the location shown in Fig. 1. Its crest is 3,008 ft above sea level and has a width of 50 ft and a length of 1,150 ft. The thickness of the embankment at stream-bed elevation is 1,200 ft. Both upstream and downstream faces have slopes of three horizontal to one vertical with 15 ft of freeboard between the crest of the spillway and that of the dam, thus making the thickness of the embankment 140 ft at normal high-water elevation. The entire upstream face is protected by a continuous reinforced concrete slab varying in thickness from 6 in. at the crest to 9 in. at the toe. This slab is monolithic with the cut-off wall, which extends along the entire upstream toe. The maximum section of the dam is shown in Fig. 2.

Stripping of the dam site, started in January 1932, consisted of removing all unconsolidated material to bedrock and as much of the rock as could be taken off by steam shovel without blasting. Material from 30 to 50 ft in depth, mostly sand and gravel, was excavated from the stream bed, and from 10 to 30 ft of soil and weathered rock was excavated from the abutments. In all, 771,000 cu yd of excavation was involved in the stripping operations, of which about 434,000 cu yd was placed in the downstream or pervious section. The remainder was placed in the diversion dam built upstream from the main dam, or was used for construction roads, or was wasted below the dam.

Two 2-cu yd electric shovels and one $1^1/_2$-cu yd steam shovel were used for stripping during the early stages, or until the abutment excavation had reached an elevation well above the stream bed. Subsequently, the $1^1/_2$-yd steam shovel, aided by a $^3/_4$-yd gas shovel, kept the stripping operations along the abutments well in advance of the building of the embankment. To ensure a clean foundation, shovel excavation was followed by hand excavation just prior to the placing of the fill. Crawler-type trucks, supplemented by tractor-drawn, crawler-type wagons, were extensively used in hauling the stripped material from the stream bed because of the steep grades to be negotiated.

PLACING THE EARTH FILL

Placing of the impervious fill, of which there are 2,125,000 cu yd in the dam, was started in June 1932 and completed in October 1933. This material was obtained from a borrow pit in the reservoir site about a mile from the dam. Three or four weeks before excavation was started the earth was uniformly wet by sprinkling to the desired moisture content. The material was well graded and was from 10 to 40 ft deep, the average shovel cut being 25 ft. Hauling was done with 8-cu yd trucks furnished under rental contract, each 2-cu yd shovel requiring about 12 trucks. With three shovels in the pit, the average output was about 10,300 cu yd per day of 16 hr. In all, about 50 trucks were used on the project.

On the dam the earth was spread with bulldozers, then rolled with sheep's-foot rollers. Eight sets of rollers were used, there being three rollers to the set. From 16 to 20 trips were required to give the proper compaction. To secure the desired characteristics in the rollers, most of them were built in the shops of the Department of Water and Power. Soil in contact with the foundation and abutments was compacted with air tampers. The pervious downstream section of the dam, comprising 750,000 cu yd, was built simultaneously with the impervious fill. Most of the material not secured from the

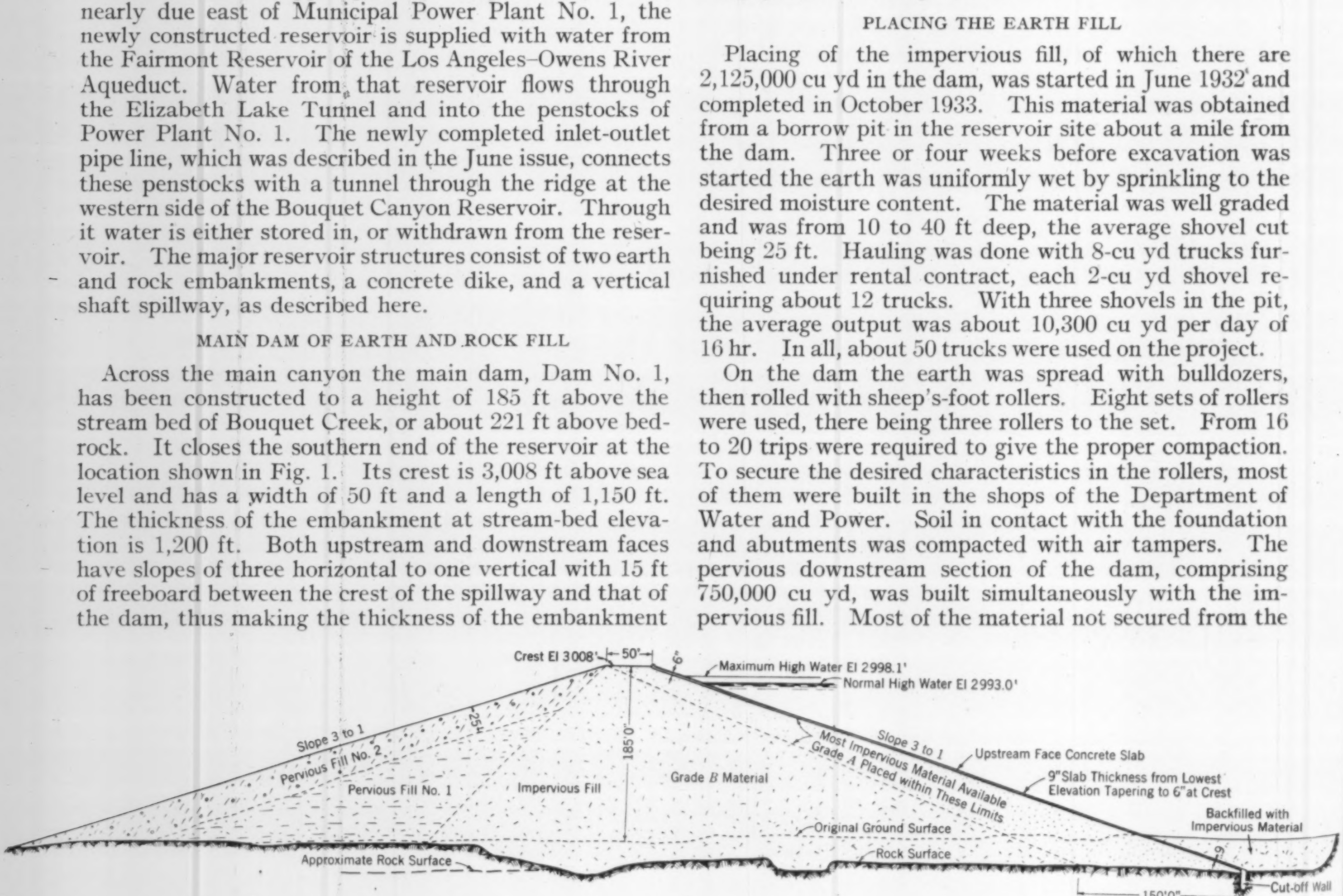

Crest El 3008'
50'
6"
Maximum High Water El 2998.1'
Normal High Water El 2993.0'
Slope 3 to 1
25'
185'0"
Pervious Fill No. 2
Pervious Fill No. 1
Impervious Fill
Grade B Material
Slope 3 to 1
Most Impervious Material Available Grade A Placed within These Limits
Upstream Face Concrete Slab
9" Slab Thickness from Lowest Elevation Tapering to 6" at Crest
Backfilled with Impervious Material
Original Ground Surface
Rock Surface
Approximate Rock Surface
9"
Cut-off Wall
150'0"
4'0"

FIG. 2. MAXIMUM SECTION OF MAIN EARTH- AND ROCK-FILL DAM

stripping operations was quarried from rock formations adjacent to, and slightly downstream from the right abutment.

At the upstream toe of the dam, a cut-off trench, varying in depth from 10 to 20 ft, was excavated and filled with concrete before the earth fill was placed. Internal air vibrators were used in placing the concrete. The reinforced concrete slab on the upstream face was built in strips 42$^1/_2$ ft wide and extending from abutment to abutment. The width of the strips was determined by the length of the reinforcing steel, which was 45 ft, of which 2$^1/_2$ ft was required for lap.

Concrete was delivered by truck from a central mixing plant to a hopper on the slope, where it was transferred to a 1-cu yd car running on an industrial track supported on timber staging. The car, which discharged into chutes at the points of placement, was pulled in either direction by an air hoist. Steel screeds were used to finish the surface, and asphaltic paint was applied for curing. The facing slab and cut-off wall together contained approximately 15,250 cu yd of concrete.

CONSTRUCTION OF SECOND DAM

To close one of the two saddles in the ridge at the western side of the reservoir, Dam No. 2 was built. It is of the same type as Dam No. 1, and its principal dimensions are as follows: maximum height above original ground surface of the saddle, 42 ft; crest width, 30 ft; cut-off wall, 2 ft 6 in. thick; and reinforced concrete face slab, uniformly 6 in. thick. It required 136,000 cu yd of fill and about 40,000 cu yd of stripping and was built in the same way as the main dam.

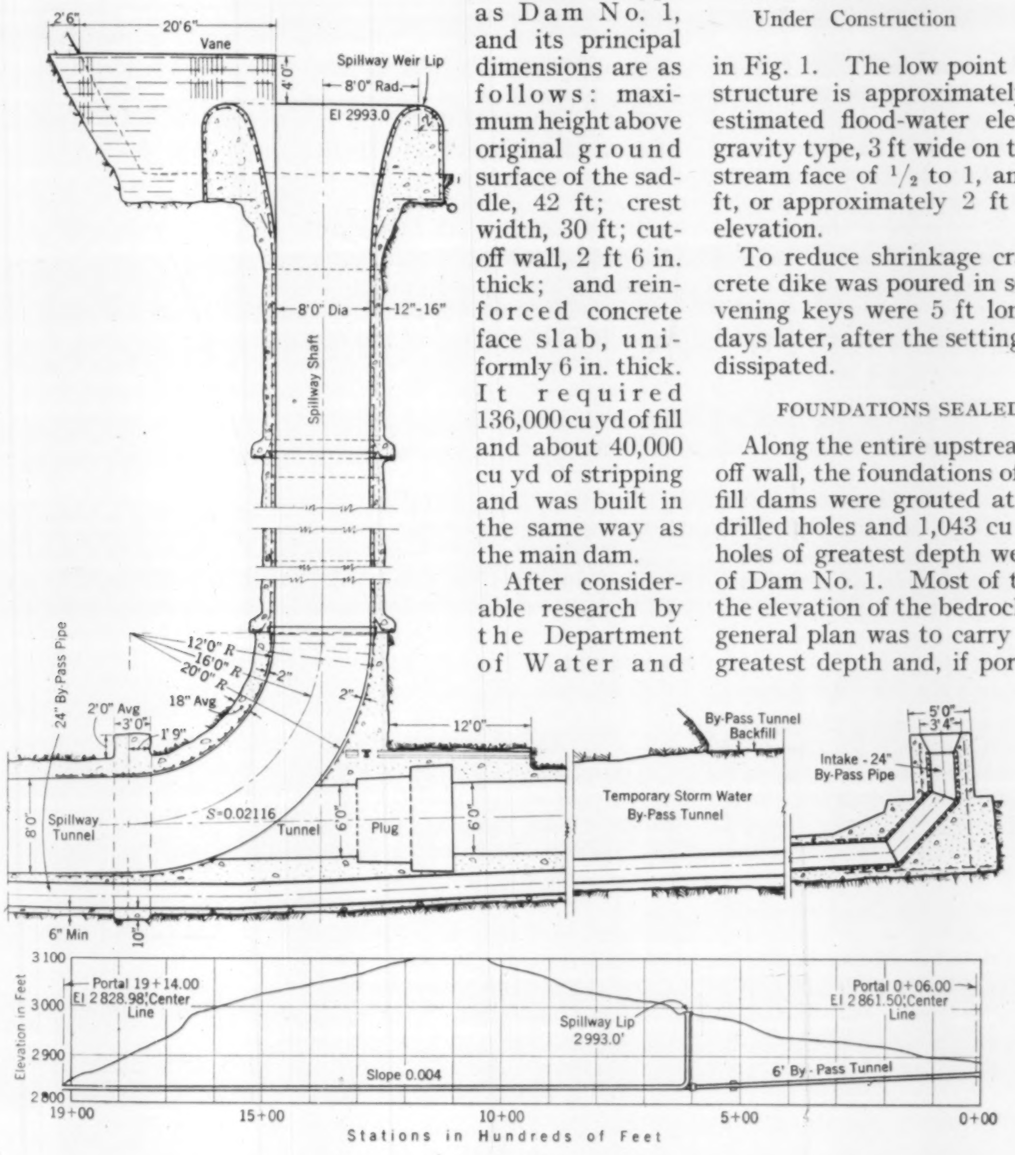
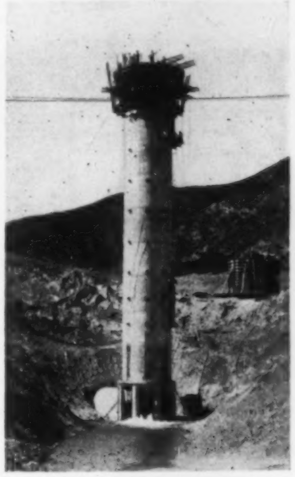

2'6"
20'6"
Vane
4'0"
Spillway Weir Lip
8'0" Rad.
El 2993.0
8'0" Dia
12"-16"
Spillway Shaft
24" By-Pass Pipe
12'0" R
16'0" R
20'0" R
2"
2"
18" Avg
2'0" Avg
3'0"
1'9"
12'0"
By-Pass Tunnel Backfill
5'0"
3'4"
Intake - 24" By-Pass Pipe
8'0"
Spillway Tunnel
S=0.02116
Tunnel
6'0"
Plug
6'0"
Temporary Storm Water By-Pass Tunnel
6" Min
10"
3 100
3 000
2 900
2 800
Elevation in Feet
Portal 19+14.00 El 2 828.98' Center Line
Portal 0+06.00 El 2 861.50' Center Line
Spillway Lip 2 993.0'
Slope 0.004
6' By-Pass Tunnel
19+00
15+00
10+00
5+00
0+00
Stations in Hundreds of Feet

Fig. 3. Profile of By-Pass and Spillway Tunnel, and Section Through Spillway Shaft

After considerable research by the Department of Water and Power, the methods of control used in placing the compacted earth fill in Dams Nos. 1 and 2 were developed. They were described by R. R. Proctor, Assoc. M. Am. Soc. C.E., in "Fundamental Principles of Soil Compaction," in *Engineering News-Record* for August 31, 1933.

The maximum and minimum percolation rates, determined by 170 percolation and consolidation tests of the rolled fill, were 0.250 and 0.005 ft per year, respectively, with an average rate for all tests of 0.010 ft per year. The maximum and minimum plasticity needle readings at the conclusion of these tests were respectively 2,000+ and 300 lb per sq in. Several auger holes and water-level manometers were placed in the fill and in the foundation below the fill to indicate the saturation line of the reservoir.



Inlet-Outlet Tower Under Construction

A concrete dike about 305 ft in length and 14 ft in maximum height was built along a saddle near Dam No. 2 at the location shown in Fig. 1. The low point of the saddle closed by this structure is approximately 6 ft below the maximum estimated flood-water elevation. The section is of the gravity type, 3 ft wide on top, with a slope on the downstream face of $^1/_2$ to 1, and the crest elevation is 3,000 ft, or approximately 2 ft above maximum flood-water elevation.

To reduce shrinkage cracks to a minimum, the concrete dike was poured in sections 25 ft long. The intervening keys were 5 ft long and were poured about 15 days later, after the setting heat of the main sections was dissipated.

FOUNDATIONS SEALED BY PRESSURE GROUTING

Along the entire upstream toe, below the concrete cut-off wall, the foundations of both the earth- and the rock-fill dams were grouted at 5-ft intervals, a total of 536 drilled holes and 1,043 cu yd of grout being used. The holes of greatest depth were required on the abutments of Dam No. 1. Most of the grout holes were carried to the elevation of the bedrock at the base of the dam. The general plan was to carry holes at 25-ft intervals to the greatest depth and, if porous rock was indicated at the lower part of these, to drill intervening holes to the same depth. Where the exploratory holes indicated sound and impervious rock, the drilling of the intermediate holes was stopped at lesser depths, determined by the nature of the rock.

All the grout holes were tested for water-tightness by measuring the flow of water into them under a pressure of 30 lb per sq in., with the exception of four holes in which the capacity of the pipe line was insufficient to maintain this pressure. These four holes also took the greatest amount of grout, which amounted to 27 cu yd in one case. Four of the holes refused to take water under pressure, and consequently no more grout was used in them than was needed to fill them. The average quantity of grout used per hole, exclusive of the amount required to fill it, was 1.95 cu yd. A total of 130 holes in Dam No. 1 and 50 in Dam No. 2 took less than 1 cu ft of grout each.

LAYING REINFORCED CONCRETE SLAB ON THE UPSTREAM FACE

A new method of grouting was employed for practically all this work. A quantity of bentonite, from 10 to 20 per cent of the cement by weight, was added to the grout mix. This had the effect of retarding the set and preventing settlement of the cement through the mixture. The grout was forced through a small pipe to the bottom of the drilled hole, whence it traveled upward. An excess of grout was pumped into the hole, permitting the withdrawal of a part of it through a valve placed in the casing at the top of the hole. In this way a continuous upward flow of grout was maintained, eliminating any difficulty from setting or settlement of cement at the bottom of the hole. When the flow of grout into the rock ceased, the entire hole was washed with water and the grouting resumed. This method permitted a continuous placing of grout, except while the hole was being washed with water, for a period of 222 hr in the hole that required 27 cu yd of grout. This grout, when set, had the appearance and physical characteristics of diatomaceous earth, but careful tests indicated that it was highly impervious to water.

DESIGN OF THE SPILLWAY

The spillway was designed to care for an estimated 0.1 per cent chance, or a "once-in-a-thousand-year" flood of 24 hr duration. The run-off for this critical period was estimated at 5,370 acre-ft and the peak flow at 6,900 cu ft per sec for the entire 13.6 sq miles of watershed above the dam. To this amount was added a possible uniform inflow of 320 cu ft per sec from the aqueduct, making a total of 6,000 acre-ft entering the reservoir in 24 hr at an estimated maximum rate of 7,200 cu ft per sec.

Assuming that the reservoir was full at the beginning of such a flood, this inflow would cause a rise of 5 ft in the water surface, which would produce a maximum spillway discharge of 1,600 cu ft per sec. Without discharge over the spillway, an accretion of 6,000 acre-ft would cause a rise of 9.1 ft above the spillway lip, and there would still be a freeboard between this lip and the top of the dam of 5.9 ft.

As shown in Fig. 3, the spillway is of the vertical-shaft type, 163 ft deep and 8 ft in inside diameter, connected to a tunnel of the same diameter with an elbow of 16-ft radius. For the upper 13 ft of its length the shaft is belled out from 8 ft at elevation 2,980 to 16 ft at the crest of the spillway weir, that is, at elevation 2,993. To prevent the formation of vortex currents around the top of the weir shaft, a radial vane extends from one wall of the shaft to the shore line at elevation 2,997. The outlet tunnel from the lower end of the shaft is 1,280 ft long and terminates in a reinforced concrete stilling basin.

A temporary storm-water tunnel connects the bottom of the shaft with the reservoir. This tunnel, in conjunction with the spillway outlet tunnel, served as a by-pass for flood waters during the period of construction of Dam No. 1. The shaft and spillway tunnel are lined with 12 in. of reinforced concrete. A permanent 24-in. concrete-lined and concrete-covered steel pipe was installed beneath the invert lining of the spillway tunnel, which extends into the reservoir through the storm-water diversion tunnel. The 24-in. pipe line will be used as a permanent by-pass for the waters of Bouquet Creek, which originate in the revervoir basin. Appropriate control valves and water-measuring devices have been placed at the outlet end of the pipe, where it discharges into the spillway stilling basin.

The shaft and tunnel were driven into schist of variable hardness, requiring a moderate amount of support. Where needed, 8 by 8-in. timbers were set on 5-ft centers. From 10 to 22 holes were drilled per round, using two drifting drills mounted on a horizontal bar. To avoid excessive overbreak or weakening of the wall rock, short rounds of 5 ft were shot with 40 per cent powder.

Tunnel muck was loaded by hand into cars of 40-cu ft capacity, which were operated by air or electric hoists. The shaft, as raised to the surface from the tunnel level, had a 6 by 8-ft cross section and was then trimmed to final size from the top down. To avoid the use of timber or supports, the placing of the concrete lining immediately followed the trimming, which was done in sections from 15 to 35 ft in depth. In a few places where support was required, sectional steel plates were employed instead of timbers.

TYPE OF FORMS USED

Collapsible steel forms were used in lining the spillway tunnel, and the same forms were used in lining the inlet-outlet tunnel. The forms were made in unit lengths of 10 ft except the invert forms, which were in 5-ft lengths. Small concrete piers, cast in advance on 5-ft centers, supported the invert forms, which in turn supported the side and arch section. Thus it was possible to pour the full circular section in one operation. The concrete was placed with compressed air.

To facilitate the excavation of the trench in the invert of the spillway tunnel to receive the by-pass pipe, and to make easier the laying of that pipe, which was welded in 30-ft sections, a monorail consisting of an 8-in. I-beam was suspended from the cap of the tunnel timbers. Muck buckets hung from trolleys transported the excavated material, and the concrete in the tunnel lining was delivered by the same means to the concrete placing "gun" located near the forms.

After completion of the shaft and spillway tunnel lining, the diversion tunnel just above the shaft was plugged with concrete and the tunnel was filled with sandy clay for 290 ft upstream of the plug. The remainder of the tunnel was filled for 3 ft over the invert to within 30 ft of the portal, where it was again completely closed with sandy clay. This material was shot in pneumatically with a gun. After completion of the lining, the inlet-outlet tunnel and the spillway shaft and tunnel for 200 ft from the shaft were thoroughly grouted with cement under a pressure of 75 lb per sq in.

As shown in Fig. 1, the gate tower is located approximately three-quarters of a mile northwest of Dam No. 1, at the western extremity of the reservoir. It constitutes the eastern terminus of the inlet-outlet conduit, a tunnel 1,027 ft long and 8 ft in inside diameter, which extends through the hills to the steel pipe line. The concrete tunnel lining has a minimum thickness of one foot and is reinforced for internal pressure. The internal diameter of this tunnel is the same as that of the spillway tunnel, thus permitting the use of the same set of steel forms for the concrete lining. The tower, 136 ft in height, constructed of reinforced concrete, is cylindrical in shape, and has a uniform outside diameter of 16 ft. A reinforced concrete penthouse at the top of the tower provides housing for the gate-operating equipment. Five cast-iron sluice gates, 42 by 60 in., of the conventional type, capable of resisting water pressure away from and towards their seats, control the outflow of water from the reservoir. Three of the gates are set at the bottom of the tower and have a sill elevation of 2,894.8 ft, which is also the elevation of the tunnel invert. Two other gates are located in the tower walls and have sills 32 ft and 64 ft, respectively, above the three lower gates. The tower was designed with a seismic coefficient of 0.1 *g*.

A bridge 276 ft long, consisting of a 200-ft suspension span and two equal shore spans, connects the tower with the shore. The suspended span is supported by two prestressed cable strands and by arc-welded structural steel stiffening trusses. An aerial cableway was used to place materials in the tower and bridge.

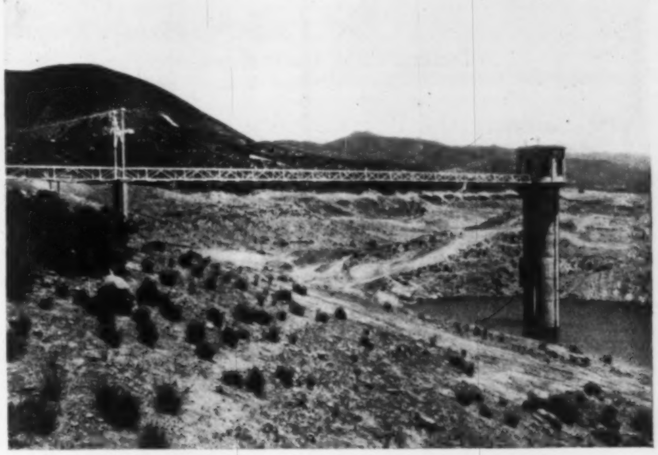
SPECIFICATIONS FOR CEMENT AND CONCRETE

The cement was certified as passing specification No. 6-9-21 of the American Society for Testing Materials and subsequent amendments, for portland cement by a test laboratory that maintains a representative at the mill at all times. The aggregate was supplied by a Los Angeles company. Tests of sand and rock deposits within the reservoir site showed these materials to be entirely suitable, but bids indicated that it would be more economical to purchase the sand and rock from a properly equipped plant than to establish the necessary field facilities to supply aggregate of the required quality.

TABLE I. COMPRESSIVE STRENGTHS OF VARIOUS CONCRETE MIXES

WATER-CEMENT RATIO Gal per Sack	YIELD Sacks of Cement per Cu Yd	FIELD MIX Loose Volume	NUMBER OF TESTS AND MEAN COMPRESSIVE STRENGTH Lb per Sq In.	
			7 days	28 days
5.0	8.5	1:1.5:2.4	(13) 3,278	(13) 4,52
6.0 to 6.5	7.1 7.6	1:2.1:2.5 1:1.8:2.2	(10) 2,180	(13) 3,887
6.5 to 7.0	5.3 6.0	1:3.0:3.6 1:2.6:3.2	(29) 2,040	(56) 3,165
7.3 to 7.7	5.60 5.2	1:2.7:3.5 1:3.0:3.6	(30) 1,797	(29) 2,754
7.7 to 8.3	4.6 5.1	1:3.2:4.4 1:3.0:4.0	(135) 1,645	(198) 2,555

Table I gives the details of the principal concrete mixes used. The variations in cement yield were principally caused by conditions of placement. All the exposed concrete was cured with an asphaltic sealing compound except where the black color was objectionable. Where the curing compound was not used, extra precautions were taken to secure thorough curing by sprinkling with water. Concrete test cylinders to the number of 642 were required to control the strength of the 27,250 cu yd of concrete placed. All the reinforcing steel was of intermediate grade and was tested in cold bend and tension to ensure its compliance with specification No. A-15-33 of the American Society for Testing Materials. Approximately 400 of each of these tests were required.



GATE CONTROL TOWER AT END OF INLET-OUTLET TUNNEL

PROGRESS OF CONSTRUCTION

All the work was done by force account, and most of it was carried out in two shifts of 8 hr each, working $5^1/_2$ days per week. The average number of men employed over a period of a little more than two years was 650. About half of these lived in the camp provided by the Department, and the rest lived in cabins in the canyons close to the work.

Slightly over two years have been required for completion of the project. Stripping of the site for Dam No. 1 was commenced in January 1932, and since that time construction has progressed continuously. The work on the other structures was started at a sufficiently late date so that they would be completed just prior to the final welding and finishing of the pipe line. Water was turned into the reservoir on March 28, 1934.

ACKNOWLEDGMENTS

Prior to and during the construction of the dams, Charles T. Leeds, Louis C. Hill, and J. B. Lippincott, all Members Am. Soc. C.E., were employed as consulting engineers. The following consulting engineers and geologists have reported on various features of the project: Charles P. Berkey, R. E. McDonnell, Thaddeus Merriman, and Allan E. Sedgwick, all Members Am. Soc. C.E.; and Robert T. Hill, F. L. Ransome, and Rush T. Sill. Construction of dams in California is subject to the approval and supervision of the State Engineer.

The entire project, including the inlet-outlet pipe line, was constructed at a cost of approximately $4,000,000 by the forces of the Bureau of Water Works under the direction and supervision of the writer, and W. W. Hurlbut, M. Am. Soc. C.E., Engineer of Water Works. Henry L. Jacques, M. Am. Soc. C.E., was Construction Engineer and Hugh Mulholland and John H. Bouey, construction superintendents. The designs and specifications were prepared by the Bureau's engineers under the direction of C. E. Angilly, M. Am. Soc. C.E. The field engineering was under the direction of R. R. Proctor, Assoc. M. Am. Soc. C.E.

Pumping Concrete for Dam Construction

Experience with New Type of Placing Equipment at Dam No. 5 on the Upper Mississippi River

By E. W. SEEMANN

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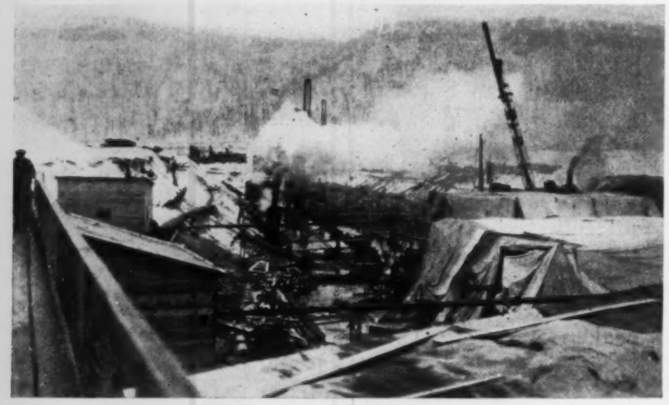
MACHINES for concrete placing operated by compressed air have been used for tunnel lining and other work in restricted quarters for a number of years. More recently a machine has been developed which successfully pumps the concrete through a pipe line by the action of a reciprocating piston in a cylinder equipped with a suitable inlet and outlet valve. In this article, originally presented as an address before the Northwestern Section of the Society, Mr. Seemann explains the use of this method on the construction of Dam No. 5 across the Mississippi River about 100 miles below St. Paul. On this job concrete having a slump of only $1^1/_2$ in. is being pumped through pipes 7 in. in diameter for distances of nearly 1,000 ft. Operation was continued during the past winter, even when the temperature dropped to ten degrees below zero. The comparatively dry and harsh concrete handled by the pumps is vibrated in the forms by the aid of electrically driven devices inserted into the mass. The satisfactory strength of test cylinders and the good quality of the concrete in place attest to the usefulness of this new type of concrete-placing equipment.

TO improve the Upper Mississippi River for navigation, the Government has begun the construction of the necessary locks and dams to maintain a navigable channel 9 ft deep. Funds for the purpose were made available through the NRA. At various points between Canton, Mo., and Red Wing, Minn., a distance of 458 miles, 17 locks and dams are either already under contract or contemplated for construction. These combination structures maintain a difference in elevation between upper and lower pools of from 5.5 to 11.0 ft. The dams will consist of movable gate sections of both roller and Tainter type. To provide for the passage of barges, each dam is to have one main lock with a clear length of 600 ft and a width of 110 ft. Foundations for the locks will be on solid rock where available, and elsewhere on piling. The upper and lower guide walls are to be supported on crib foundations. The upstream gate bay and gate for a future auxiliary lock are being constructed as a part of the main lock contracts for the various dams.

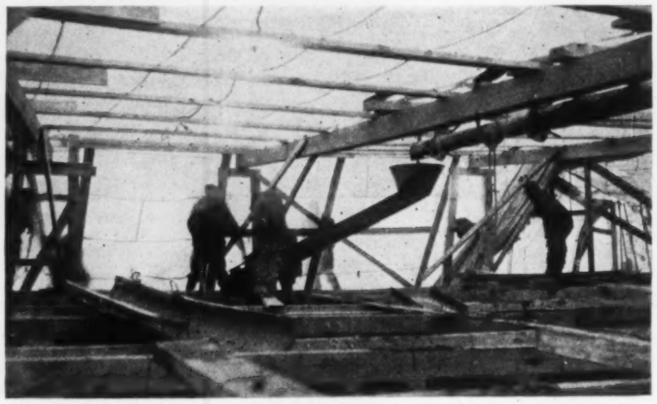
Twelve miles upstream from Winona, Minn., near Fountain City, Wis., Dam No. 5 is now under construction. When finished it will consist of a movable gate section in the main channel of the river, adjoining the locks on the Minnesota bank, now completed, and an earth-fill dike section extending to the Wisconsin bank. There will be a difference of 11 ft in the elevation of the upper and lower pools. The movable gate section consists of 6 steel roller gates and 28 steel Tainter gates with an overall length of 1,619 lin ft, supported on foundations of timber and steel piling. The roller gates will be 60 ft long between concrete piers, 15 ft in diameter, and 20 ft high above the sills. They will be operated by fixed electric hoists housed on the tops of the piers. The Tainter gates will be 35 ft long between concrete piers and 15 ft high above the sills and will be operated by traveling electric hoist units mounted on the service bridge.



WINTER CONSTRUCTION OF COFFERDAM NO. 1
Pipe Line Laid to Pump Concrete Into Protected Forms

A steel service bridge extending along the upstream side of the dam will carry a 20-ton gasoline locomotive crane for use in handling drift and ice and in placing the emergency bulkheads for closing the openings downstream between the piers for the roller gates. To hoist the Tainter gates and handle the upstream emergency bulkhead units, four traveling hoist units will be provided on the bridge, three for electric operation and one for gasoline operation. A fill supporting tracks and other facilities will be constructed adjacent to the abutment of the movable-gate section. This fill will be used for the storage of emergency bulkheads.

To give an idea of the magnitude of the work, some of the quantities included in the permanent construction follow: 1,665 lin ft of work inclosed in cofferdams; 86,500 cu yd of common excavation; 33,000 cu yd of fill; 170,000 lin ft of round timber piling; 4,700 l.n ft of concrete piling; 107,000 sq ft of steel sheet piling for cut-off walls; 14,000 cu yd of riprap rock; 45,000 cu yd of concrete; 375 tons of reinforcing bars; 2,200 tons



PIPE LINE DISCHARGING CONCRETE TO PROTECTED FORMS

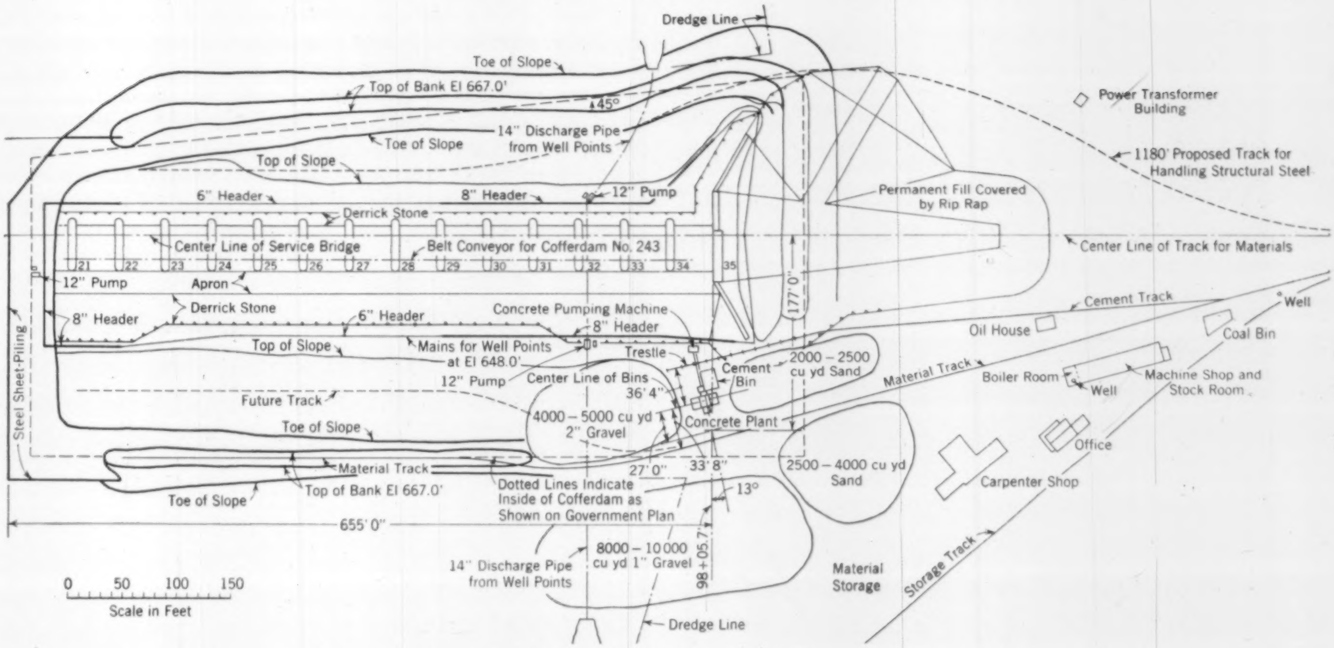

Dredge Line
Toe of Slope
Top of Bank El 667.0'
45°
14" Discharge Pipe from Well Points
Toe of Slope
Top of Slope
6" Header
8" Header
12" Pump
Derrick Stone
Center Line of Service Bridge
Belt Conveyor for Cofferdam No. 243
21
22
23
24
25
26
27
28
29
30
31
32
33
34
35
12" Pump
Apron
8" Header
Derrick Stone
6" Header
Concrete Pumping Machine
8" Header
Top of Slope
Mains for Well Points at El 648.0'
Trestle
12" Pump
Center Line of Bins
36' 4"
Cement Bin
2000 – 2500 cu yd Sand
Steel Sheet-Piling
Future Track
4000 – 5000 cu yd 2" Gravel
Concrete Plant
Toe of Slope
Material Track
27' 0"
33' 8"
2500 – 4000 cu yd Sand
Toe of Slope
Top of Bank El 667.0'
Dotted Lines Indicate Inside of Cofferdam as Shown on Government Plan
13°
655' 0"
177' 0"
14" Discharge Pipe from Well Points
8000 – 10000 cu yd 1" Gravel
98 + 05.7'
Material Storage
Dredge Line
0 50 100 150
Scale in Feet
Power Transformer Building
1180' Proposed Track for Handling Structural Steel
Permanent Fill Covered by Rip Rap
Center Line of Track for Materials
Cement Track
Well
Oil House
Coal Bin
Material Track
Boiler Room
Well
Machine Shop and Stock Room
Office
Carpenter Shop
Storage Track

Fig. 1. Construction Plant and Cofferdam No. 1 at Dam No. 5

Location of Well-Point Header Lines, Central Mixing Plant, and Concrete Pumping Machine Indicated; Also Tracks and Belt Conveyor Lines for Transporting Materials and Concrete to Cofferdams Nos. 2 and 3

of steel, exclusive of the roller gates; 67 tons of forgings; and 170 tons of steel castings.

The approach to the site is on the Wisconsin side of the river over about $2^1/_2$ miles of bottom land. Through this area a 100-ft right of way was acquired, cleared, and graded to accommodate a railroad and roadway for vehicular traffic. In Fig. 1 is shown the layout of the construction plant for cofferdam No. 1.

UNWATERING THE COFFERDAMS

To accomplish the construction, the site has been divided into three nearly equal parts, each enclosed in a cofferdam about 600 ft long and 300 ft wide. The first cofferdam, No. 1, encloses the easterly third of the dam; the second, No. 2, the adjoining central third in the middle of the river; and No. 3, the remaining part adjacent to the locks on the west bank. Concreting in cofferdam No. 1 has been completed; and concreting in cofferdam No. 2 is approaching completion. Cofferdam No. 3 has not yet been started. Virtually all the excavation in cofferdams Nos. 1 and 2, enclosing about 1,200 ft of the east end of the dam, was accomplished by dredging before the outer ends of the cofferdams were closed with sheet piling.

As the sand underlying the work extends to an approximate depth of 200 ft, it was elected to unwater the cofferdams by means of well points. Each of these points is 22 ft long and has a steel two-port jetting shell at its lower end, which enables the point to be jetted into the ground by water pressure delivered at the top of the well point. The shell has two valve ports controlled by floating-ball checks. When jetting, one valve port closes to prevent the water from escaping through the screen, while the other opens to allow it to discharge at the jetting tip. When suction is applied, the valve at the tip closes to prevent the entrance of sand, while the other opens to admit water from the screen. This action is automatic.

A pipe of the required length is welded to the valve shell, and a wooden backing piece is banded to the pipe, the lower end of which is slotted. Around the slots the screen is wrapped, banded, and firmly secured. On each well point, 270 sq in. of mesh permit the water to enter and pass downward into the shell, whence it is drawn up through the pipe.

In cofferdam No. 1, the 360 points were spaced 5 ft apart and were connected to 6-in. and 8-in. headers. Three 12-in. centrifugal pumps, electrically driven and connected to the headers, supplied the suction. During the dewatering period the three pumps were discharging from 5,000 to 6,000 gal per min each. An interesting point is that the temperature of the water pumped out remained at about 50 F irrespective of the temperature of the air.

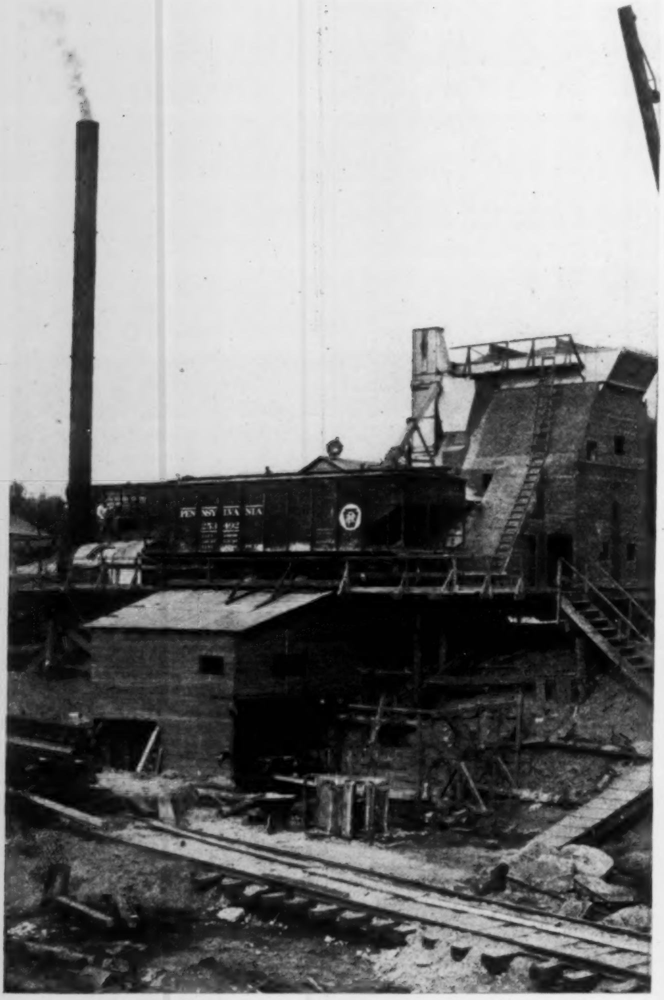
CONCRETE CHARACTERISTICS

Specifications for the cement are standard, except that the residue on a 200-mesh sieve is required to be not less than 15 per cent. This specification results in a concrete that generates less heat in setting than more finely ground cement. To obtain the specified average strength of concrete of 3,200 lb per sq in., the coarse aggregate is separated into two sizes, that measuring up to 1 in. and that measuring from 1 to 2 in. At least $4^1/_2$ bags of cement per cubic yard of concrete are required, and a maximum water-cement ratio of $6^1/_2$ gal per bag of cement is permitted.

By trial it was found that, with the materials available, a mix consisting of 37 per cent sand, 63 per cent coarse aggregate, and $4^1/_2$ bags of cement per cubic yard of concrete could be placed with the use of vibrators. In the field the mix runs about in this proportion and has a cement yield of about 4.5 to 4.6 bags per cu yd of concrete and a water-cement ratio of about 5.8 gal per sack of cement. Concrete cylinders have shown strengths up to and better than 4,000 lb per sq in. in the testing machine.

This concrete is extremely harsh, unlike that generally understood to be a good placeable mix. The slump averages from 1 to $1^1/_2$ in., and the concrete cannot be placed by the usual methods.

Therefore the specifications provide for the use of



CENTRAL CONCRETE MIXING PLANT
Dual Concrete Pumping Machine in Shed Under Trestle

vibrators. Three of these are used, not attached to the forms but placed in the concrete mass, which materially changes its appearance when settled into place by this means. Its harshness disappears and it becomes quite fluid.

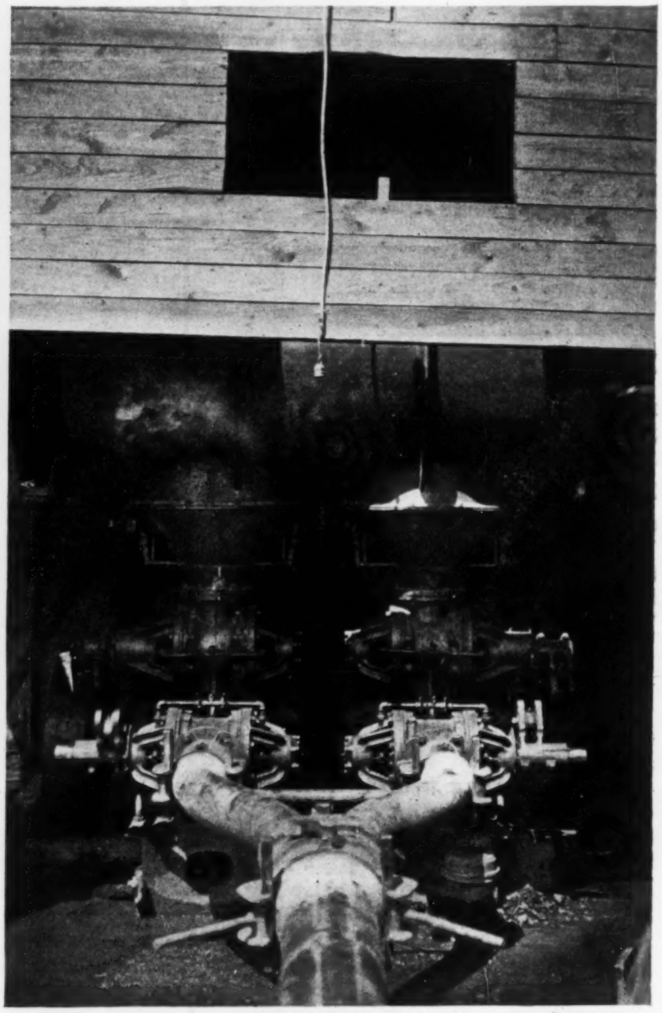
A vibrator consists of an electric motor encased in a steel shell mounted on a pipe handle. The motor is mounted slightly eccentric and rotates within this shell, which is inserted into the concrete mass. Good surfaces, with very little honeycombing, are obtained without the use of the familiar concrete spade.

AUTOMATIC DEVICES AT CENTRAL CONCRETE PLANT

The concrete plant consists of two 1-cu yd mixers, electrically operated and fed by three charging hoppers, one for sand and one for each of the two sizes of aggregate. From the charging hoppers the aggregate is discharged into the weighing hoppers. The mixers are set back to back so that a two-way chute with a gate permits the discharge of the aggregate into either mixer. Time relay switches on both mixers ensure the proper mixing time. Each mixer is equipped with a counter as is also the cement weighing hopper. In this way an accurate record of the number of batches is obtained.

Cement arrives at the mixing plant in bulk, and a screw conveyor delivers it from the car to an auxiliary hopper. Since the capacity of the unloading screw is greater than the requirements of the plant, the excess overflows into a storage bin from which it is again sent up to the auxiliary hopper when the plant is not drawing direct from a car. The cement is automatically weighed.

There are 12,500 cu yd of concrete in cofferdam No. 1. During the fall of 1933 aggregate for this amount was placed in stock piles, which for winter operation were kept warm by means of steam jets. Aggregate in the hoppers was heated by an arrangement of steam-pipe grids, and the water was heated before it entered the mixer. For the protection of the newly placed concrete, 5,300 sq yd of canvas and 2,700 yd of paper made of sisal built into panels were used on the work. Salamanders, steam pipes, and electric grids were employed for heating.



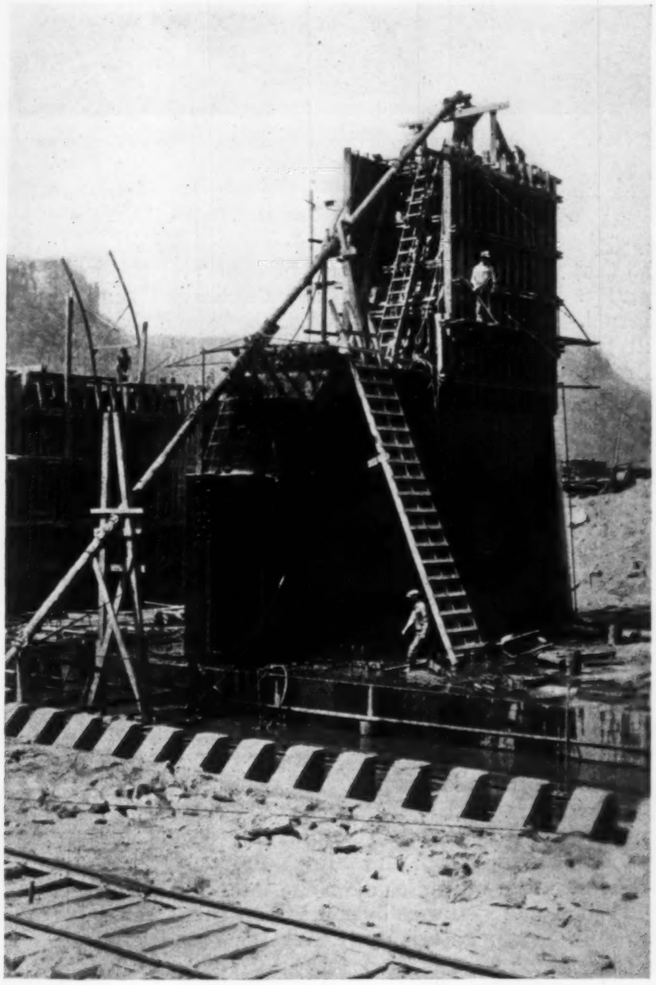
DISCHARGE END OF CONCRETE PUMPING PLANT

CONCRETE PLACED BY PUMPS

After various methods of conveying the concrete to the forms had been considered, it was decided to use a combination of belt conveyor and concrete pump. The concrete is handled in the pump machine in very much the same way that water is in a piston pump. The concrete pump is of course especially designed throughout for its purpose. It consists of a nearly conventional collection of crank shafts, connecting rods, pistons, and cylinders (Fig. 2). The valves are large plug cocks, mechanically opened and closed in synchronism with the movements of the piston. On the suction stroke the concrete passes through the inlet valve into the pump cylinder and completely fills it. At the end of the suction stroke the inlet valve is closed and the outlet valve opened, so that on the pressure stroke the concrete is forced through the outlet valve into the pipe line, and all the concrete in the pipe is moved forward. After the pipe line is once filled, the amount of concrete forced out of the end at each stroke is naturally equal to that forced through the outlet valve of the pump by the piston.

These concrete pumps are set in pairs similar to a duplex water pump, each piston alternately delivering its charge of concrete to the pipe line through a Y-connection. Either one or both cylinders are used, depending somewhat on the rate of pouring. The capacity of a duplex pump varies from 25 cu yd per hr for single action to 50 cu yd per hr for duplex operation. Each cylinder has a bore of $7^1/_2$ in. and a stroke of 12 in. With the electric drive used at Dam No. 5 the pumps operate at about 50 strokes per minute.

All this seems very simple, but actually a number of difficulties must be overcome before such a pump can function effectively. First, concrete, more than any other fluid, objects to doing anything except staying where it is or moving in a straight line, preferably downhill. If left in the pipe for any length of time it hardens. Second, it is highly abrasive. Third, there are times when it can hardly be classified as a fluid. This applies particularly to the work at Dam No. 5.



Concrete Pipe Line to an Upper Form of Pier

These peculiarities of concrete have resulted in some interesting details in the design of the pump. For example, the valve ports and the pipe line are nearly as large in diameter as the cylinder itself. There are no sharp corners or sudden changes of direction at any point, either in the pump or in the pipe line. Concrete has a peculiarity of packing where there is a sudden reduction in the size of the passage through which it is forced. This characteristic has been used to advantage in the design of the valves. Instead of closing fully, they normally close only partially, since the concrete will pack at the restriction and complete the closure. This obviates the necessity of having the valves act as rock crushers or stone cutters, and consequently the cause of greatest wear and tear is eliminated. The valves are adjustable to close either partially or entirely, depending on the size of the aggregate. Springs in the connecting rods prevent damage to the machine in case the valve closes on a rock larger than it is set for.

Concrete has been pumped through a 7-in. pipe line for a horizontal distance greater than 1,000 ft. With very dry concrete, an equivalent length of about 800 ft is about the limit of good practice. In determining the equivalent length, an allowance of 40 ft is made for each 90-deg elbow. At Dam No. 5, the greatest distance concrete has been pumped so far has been about 1,000 ft of equivalent length. Concrete as dry as that having a 1-in. slump has been pumped, but in working with consistencies less than those of 2-in. slump it is imperative that a very careful control be maintained in batching

Hood Over End of Pipe Diverts Concrete to Chute and mixing and also in proportioning the various ingredients. It is ordinarily safe to interrupt pumping for half an hour, but generally a few strokes are pumped every few minutes during an interruption to keep the pipe line clear.

MIXING PLANT AND CONCRETE PUMPING LINES
Well-Point Suction Line Beside Concrete Pipe Line

Pipe sections are connected with special toggle couplings that can be connected and disconnected in a few minutes. The standard length of a section is 10 ft. With the addition of 5-ft, 3-ft, and 1-ft straight sections and 90-deg, 45-deg, and $22^1/_2$-deg bent sections, for use in combination as required, almost any desired point can be reached.

A signal system of electric lights is in operation between the forms and the concrete pumping plant, and between the pumping plant and the mixers.

CLEARING THE PIPE

Of interest is the procedure followed as the end of a pour is approached. The foreman at the forms knows from the number of pipe sections just how much concrete there is in the pipe. When he has this amount yet to place he signals the man at the concrete pump to clear the pipe line. One or two sections nearest the pump are removed and a "go-devil" is inserted in the line. This go-devil is a dumbbell-shaped piston with a special rubber ring on each end which prevents the water from getting past into the concrete. One unit of the dual pump is closed, and the end of the pipe is then connected to the other cylinder, which acts as a water pump, forcing the go-devil along ahead of the water. The concrete in the line is thus pushed into the forms, and at the same time the pipe is washed by the water. The exact location of the go-devil can be determined at any time by tapping the pipe. When the go-devil starts its journey, a man walks along the line and keeps track of its progress. Just before it reaches the end of the pipe, a chute is placed so that the last small amount of concrete, the go-devil, and a little of the water are discharged outside the forms. The signal is then given, the pipe is disconnected at the concrete pump, and the pipe line is allowed to drain. No further washing of the pipe is necessary.

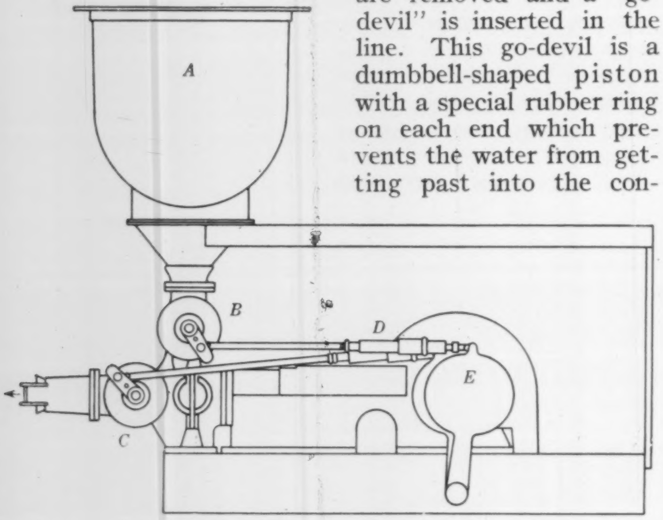
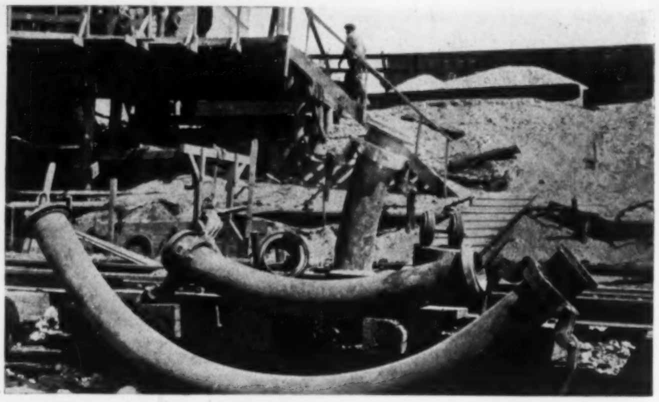


FIG. 2. DIAGRAM OF CONCRETE PUMP

A, Receiving Hopper with Agitator; *B*, Inlet Valve; *C*, Outlet Valve; *D*, Enclosed Spring in Connecting Rod; *E*, Eccentric which Operates Valves. In Operation, Valve *B* Opens and Concrete Drops Into Chamber Below; Valve *B* Closes and Valve *C* Opens as the Piston Moves Forward, Forcing the Concrete Into the Pipe Line. The Concrete in the Line Moves Forward a Distance Equal to the Length of the Piston Stroke



TYPES OF BENDS AND COUPLINGS USED IN CONCRETE PUMPING PIPE LINE
The Dumbbell-Shaped Piston or "Go-Devil" Separates the Wash Water from the Concrete at the End of a Pour

Placing concrete by pump and pipe line was found to have some advantages not available with other methods. In cold weather, the pipe line was successfully insulated to prevent too much cooling of the concrete. Also, the area at the forms in need of protection was reduced to a minimum. The pipe lines were placed in advance for the various pours so that very little time was lost in completing one monolith and starting another, even at a considerable distance away. When pipe lines are used there is less congestion at the site than with any of the usual methods of placing.

Although the concrete pump is well developed, the arrangement of chutes and the manner of economically handling the concrete at the discharge end of the pipe requires considerable study to increase the efficiency of this method of placement. The concrete being placed on this work is so stiff that occasionally a batch plugs the machine. Fortunately this plugging occurs at the taper section leading from the pump, and only a few minutes of work are required to open the joint and clean it out.

When the distance becomes too great for successful pumping, a booster pump may be installed at the end of the line. On the work at Dam No. 5 a belt conveyor combined with automatic dump cars running on industrial track is contemplated. By this means the concrete would be conveyed to points in cofferdam No. 3 whence it can be successfully relayed to the forms by the concrete pumps.

On July 1, a total of 1,166 lin ft of permanent work had been enclosed in cofferdams, and 20,000 cu yd of concrete had been pumped. Also, 20 Tainter gates had been put in place, and bridge girders had been set over 17 gate openings. More than 50 per cent of the total work had been completed.

Clay Stratum Dried Out to Prevent Landslips

Heated Air Forced Through Tunnels aad Drill Holes to Control Earth Movement

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IN the fall of 1932 two landslides brought 100,000 cu yd of earth crashing down from a bluff on to a heavily traveled state highway along the Pacific Ocean northwest of Santa Monica, Calif. Not only was the highway rendered temporarily impassable but a highly improved residential estate on top of the 180-ft bluff was considerably damaged. To prevent further slips, which might undermine or destroy the principal building on the property, a unique control system was built and put into operation. A network of interconnected tunnels and drill holes was driven into the clay stratum at the base of the bluff. For the past year, air heated by a specially built natural-gas furnace has been blown through the network to dry out the clay and increase the coefficient of friction along the slickensided stratum. The capitalized cost of operation of the blower, which may remain in use indefinitely, is considerably less than the cost of any permanent construction that could be built to control or stop possible future movement of the bluff.

WEST of Los Angeles, the slopes of the Santa Monica Mountains terminate abruptly at the ocean shore, forming what are known as the Palisades. A short distance down the coast from Santa Monica the bluff at the end of the slope merges with the coastal plain. Up the coast, along the north shore of Santa Monica Bay, the bluff becomes progressively higher, until it merges into the steep slopes of the mountains, which there come very close to the ocean.

One of the larger estates on the Palisades a short distance west of Santa Monica is shown in the aerial photograph. At this point, where the bluff is approximately 180 ft in height, two major landslides occurred in the fall of 1932 and removed half the grounds in front of the main building of the estate.

FIRST SLIDES OCCUR

The first of these slides, which occurred on September 29, brought down about 40,000 cu yd of material from the area just to the east of the main building. While this was being cleaned off the highway, it was noticed that movement was taking place to the west in a clay stratum several feet above the level of the pavement. On October 2 this section slid out and carried with it an additional 60,000 cu yd of the bluff. Fortunately for purposes of observation, the second slide occurred in the middle of the afternoon within a few minutes of the time estimated. This made it possible to measure the outward movements which preceded the slide and to take a complete motion picture of the slide itself. The situation after both the slides took place is shown in Fig. 1 and in the aerial photograph.

For more than 100 ft along the face of the bluff, the outward movement of the entire mass was pronounced, and an overhang of more than 8 in. was measured. From a study of the motion pictures it was concluded that probably an outward movement of from 1 to 2 ft took place before there was much more than moderate cracking of the face of the bluff. At the base of the slide the movement was almost horizontal, but at the top the displacement was substantially all downward. In fact at one point a section of the garden about 40 ft square came to rest over 20 ft below its former level without tipping over objects of little stability.

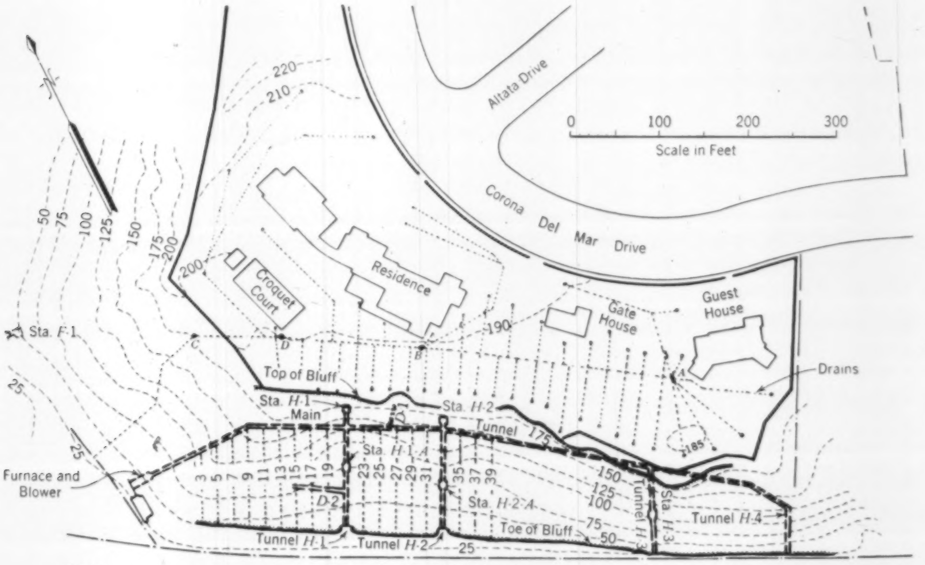
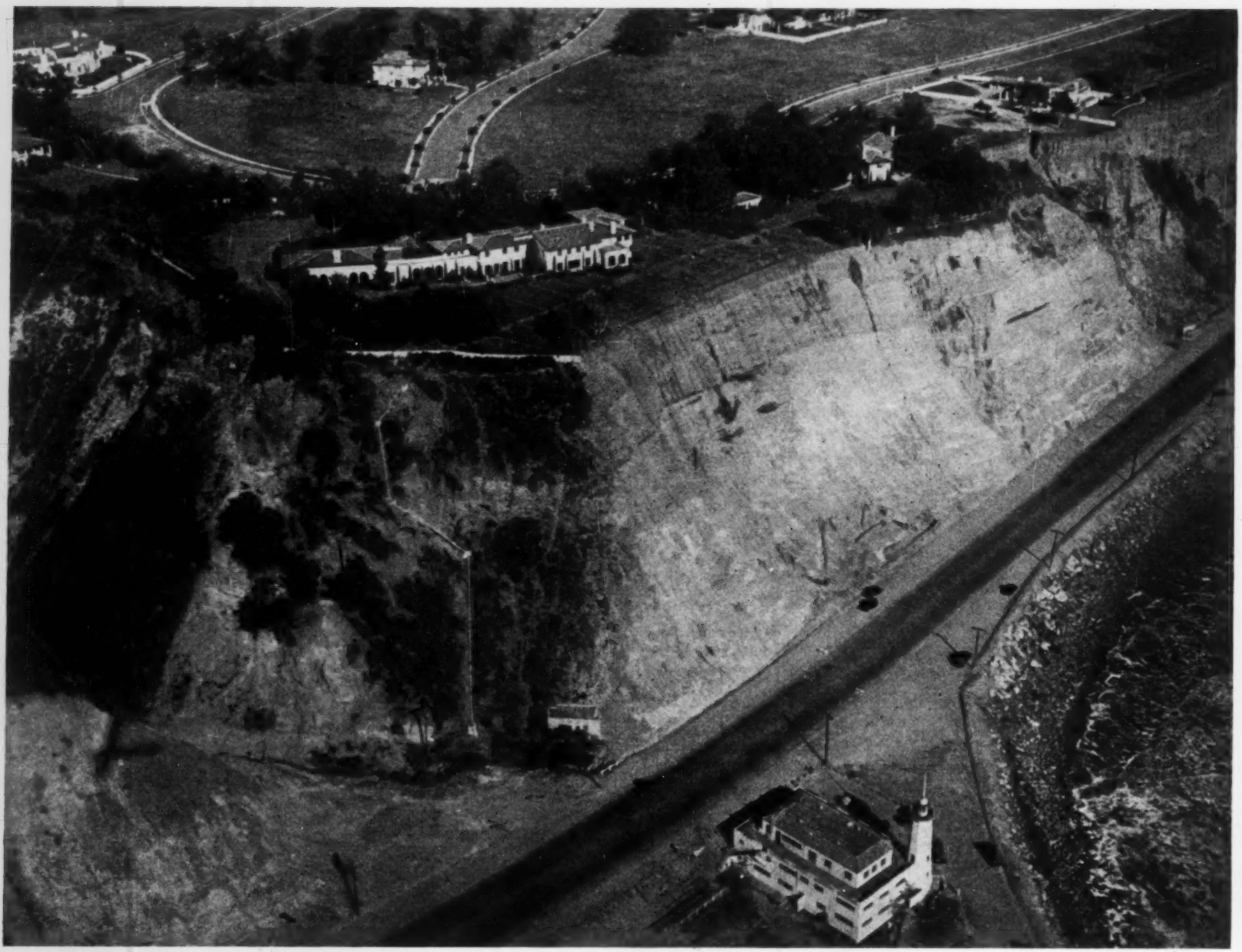


FIG. 1. MAP OF PROPERTY AFFECTED BY THE LANDSLIP
With Location of Tunnels, Drill Holes, and Subsurface Drains

Along the base of the bluff the highway had recently been widened and the new slopes had been carefully trimmed; consequently, the manner in which cracks developed in the mass could be observed with accuracy. At first a series of diagonal cracks was formed, tending generally toward the west edge of the slide. These were followed by cracking in the opposite direction. There was then a bulging of the face for some considerable distance above the base on which the outward movement was taking place. Almost immediately afterward the mass broke up, and in a few moments the entire highway was covered with debris.

No slides of any consequence had occurred along this part of the Palisades for a great many years, but this was no assurance



Extensive Landslip Threatens Valuable Estate near Santa Monica, Calif.
Aerial View of Slide, Highway, and Damaged Property

that the two slides mentioned might not be forerunners of others. Should there be another slide in front of the main building of the estate, there was the possibility that it would break back to the building itself and even might undermine it. A very thorough geologic examination was therefore made by Harry R. Johnson for the owners of the property.

He found that the slides had taken place on slickensides in a nearly horizontal stratum of blue clay about 10 ft above the level of the highway. About 500 ft to the east of Potrero Canyon, shown in the foreground of the aerial photograph, the clay stratum is cut off by a mass of cemented gravel, which may have been part of an ancient stream channel, and it extends only a short distance west of Potrero Canyon, where it is cut off again by the Potrero Fault.

Along the Potrero Fault, which strikes northeast across the grounds of the estate, there has been a total vertical displacement of approximately 150 ft. This has been progressive over a great length of time, as indicated by lessening amounts of displacement of successive strata, which in the more recent formations comprising the top of the plateau amounts to less than 10 ft. At the base of Potrero Canyon the fault dips about 45 deg to the northwest, but a short distance above this level it divides, and one branch is substantially vertical. The maximum movement has occurred on this vertical branch, although the clay stratum and other beds are deflected upward more noticeably against the inclined branch of the fault. Between the branches of the fault a wedge has thus been formed, which in the opinion of the geologist might be producing considerable thrust outward toward the ocean. Some such explanation seems almost necessary to account for the great outward movement, on a substantially horizontal clay stratum, which preceded the slides of 1932.

SHAPE OF SLIP AND POSITION OF CLAY STRATUM

Originally the face of the cliff had a slope of about 0.75 to 1 at the rather characteristic location for which a profile is given in Fig. 2. When the highway was widened, the toe of the slope was cut off, as shown, and this probably was the immediate cause of the slides. The position of the clay stratum was disclosed by the exploratory tunnels that were driven. The drift, D-2, excavated along the line of fracture at the level of the clay stratum, definitely determined the position of the back of the slide. The line of the break at the top of the bluff was of course apparent, and after the slide the surface of rupture was exposed for about 60 ft below the top.

When the loose material was removed, the top of the bluff was cut back to a more stable slope, also illustrated in Fig. 2. Underlying the loose material there was found to be a wedge of relatively unbroken ground, which however had been displaced downward from 10 to 20 ft, as evidenced by characteristic strata. This material was left in place as a supporting toe. It is significant that the slides were thickest at the base. At the point illustrated in Fig. 2 the line of fracture was 90 ft back from the original face and 70 ft back from the face that was left after the highway was widened.

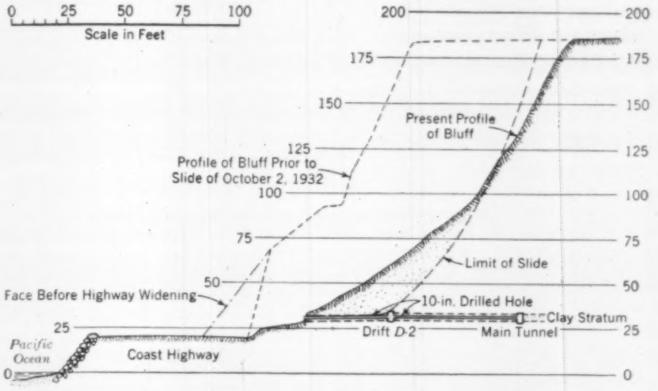


FIG. 2. CHARACTERISTIC PROFILE OF BLUFF, SHOWING EXTENT OF SLIDE, AND TUNNEL AND DRILL HOLE FOR DRYING CLAY STRATUM

It was observed that the blue clay at the base of the landslips contained a considerable amount of moisture. The slickensides within the clay were very pronounced, and conditions were undoubtedly favorable to further movement. The decision was immediately reached, therefore, that tunnels should be driven following the clay layer to collect any free water and also to determine whether slickensides were present in the clay behind the slide.

Two exploratory tunnels were started in the face of the cliff, one near the center of each of the larger slides. These tunnels were extended back about 70 ft beyond the surface of rupture, and a cross cut was driven. While no free water was found, the clay was quite damp, showing a loss in weight of from 20 to 25 per cent when dried. Accordingly, additional tunnels were driven, as shown in Fig. 1, from the Potrero Canyon along an approximate arc located well behind the slides to a point on the face of the bluff along the highway near the east line of the property. An additional adit, H-3, was driven following the clay near where it was intercepted by the cemented gravel in the old channel.

Throughout the entire length of these tunnels slickensides were found in the clay, but again no free water was encountered except in the cemented gravel, whence a small amount came out through fissures. Moreover, during the time required to drive the tunnels, definite movement was observed along the slickensides, and this was not restricted to the zone in front of the surface of fracture of the slides of 1932.

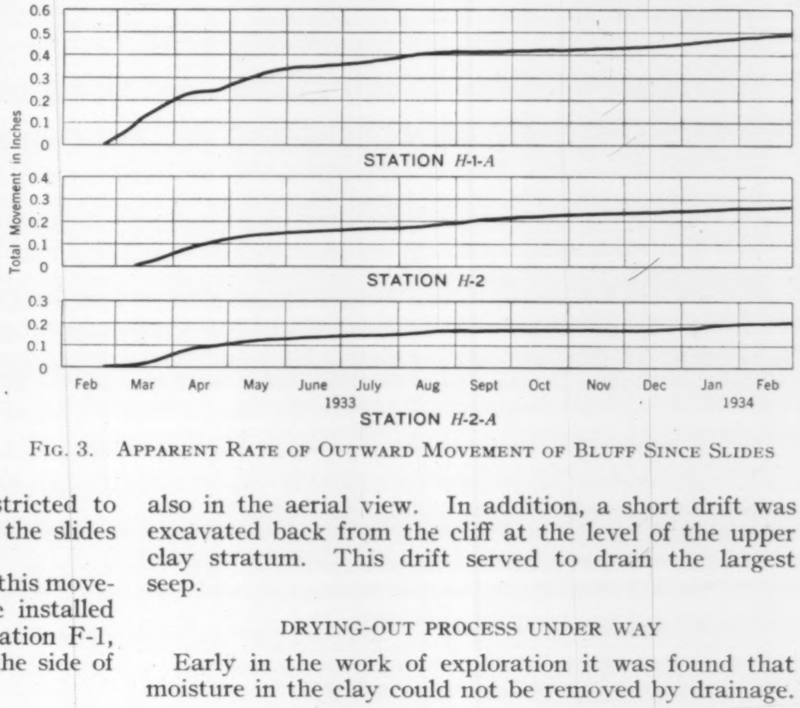


FIG. 3. APPARENT RATE OF OUTWARD MOVEMENT OF BLUFF SINCE SLIDES

In order to determine exactly the amount of this movement, stations for precise measurement were installed at five points in the tunnels, and another, Station F-1, was placed on the other side of the fault in the side of Potrero Canyon.

At each station eight measuring points were provided, consisting of ground steel balls attached to 3-ft lengths of drill steel, which had been driven into the material surrounding the tunnel. These points might be considered as being roughly at the corners of a 4-ft cube; at the four lower points the drill steel was driven vertically into the clay and underlying material below the slickensides, and at the four upper points the steel rods were driven horizontally into the material above the slickensides. Precise micrometers, permitting the measurement of distances between these points along the edge of the cube and diagonally across the face of the cube were specially built, and a precision level was constructed to determine any differential change in elevation of the four base points. With the use of each of these instruments, measurements to one-thousandth of an inch could be made. Such measurements have been taken at approximately weekly intervals for almost a year.

OUTWARD MOVEMENT MEASURED

In Fig. 3 the approximate amount of outward movement is shown, as measured at three of the stations. It will be noted that for several months early in 1933 there was movement at Station H-1-A amounting to approximately 1/10 in. per month. This station is about 75 ft back of the face of the October 1932 slide and is in presumably stable material. A similar movement, but of lesser magnitude, has taken place at the two stations in Tunnel H-2, Station H-2-A being in the same relative position as Station H-1-A, and Station H-2 being at the extreme back end of Tunnel No. 2. During the summer of 1933 the outward movement at these stations gradually lessened, and this decrease has generally continued.

About 100 ft above the clay stratum at the base of the slides, there is another layer of clay. Water percolating downward from the surface is largely intercepted by this stratum and moves horizontally towards the face of the cliff. Some of the seeps thus formed were large enough to keep the material below fairly wet. In order to intercept as much of this percolating water as possible, a system of tile drains was installed throughout the grounds of the estate (Fig. 1). The tile lines are visible also in the aerial view. In addition, a short drift was excavated back from the cliff at the level of the upper clay stratum. This drift served to drain the largest seep.

DRYING-OUT PROCESS UNDER WAY

Early in the work of exploration it was found that moisture in the clay could not be removed by drainage.

After consultation between the engineers and geologists, the decision was reached that the forces causing movement of this great mass could not be resisted satisfactorily by any retaining wall or other barrier which could be erected at reasonable cost. It was the consensus of opinion that the likelihood of further slides could be greatly minimized, if not obviated, by drying out the lower clay stratum so as to increase the frictional resistance to movement. It had been found that, when samples of the clay were dried, they changed their character markedly, becoming quite hard, and that the slickensides, which originally were polished faces, became checked. Various methods of removing the moisture were considered, but the cheapest and most satisfactory seemed to be the utilization of hot air to serve the dual purpose of heating the mass and evaporating the moisture contained in the clay. Accordingly a furnace was placed in the Potrero Canyon portal of the main tunnel. This furnace was designed to raise the temperature of a large volume of air to be forced into the tunnel by a blower. It might have been somewhat more efficient to have kept the air from mixing with the gases of combustion, but this would have required a much more expensive installation and a large part of the heat would have been lost with the flue gases. The arrangement of the blower and furnace is given in Fig. 4.

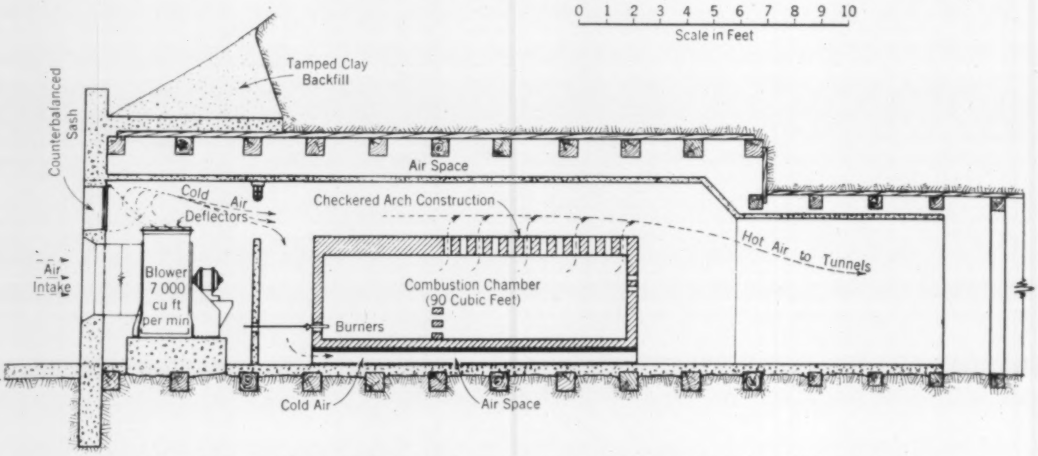


FIG. 4. ARRANGEMENT OF FURNACE AND BLOWER AT WEST PORTAL OF MAIN TUNNEL
Apparatus for Supplying Hot Air to Tunnels

The blower draws air from the outside and discharges directly into the furnace room. As there is no other outlet except through the tunnel, the cool air is forced over and around the furnace in which the gas is burned. The amount of air handled by the blower is approximately 6,500 cu ft per min, and the amount of gas burned is about 400 cu ft per hr. A thermostat, some distance down the tunnel, is set so that the air is heated to a temperature of approximately 135 F, which represents generally a rise of about 70 F above the normal air temperature.

Special protective devices are provided for the furnace, which operates automatically. If the electric current should fail, the blower of course would stop. Accordingly the flow of gas is controlled by a regulator, which completely shuts off the supply of gas, except for the pilots, in such an event. In order to prevent the temperature in the furnace room from rising to a dangerous point, the window at the end near the blower is fitted with a counterweight so arranged that when the stream of air from the discharge of the blower is not pressing against the window, it swings open. Until this rather crude provision was made, temperatures as high as 270 F were reached at times in the furnace room due to radiation from the brick of the furnace.

In case the gas supply is temporarily shut off, the thermostatic pilot with which the furnace is equipped stops the entire flow of gas, and the furnace must be restarted manually. This prevents a mixture of gas and air from being carried into the tunnels, with the resulting danger of a serious explosion.

A short distance down the tunnel from the furnace, a fire door has been installed so that in case of an accident due to possible failure of any of the other protective devices, or to any other cause, the main system of tunnels will be automatically closed off from the furnace room. The maximum damage will thus be limited to the burning of the timbers in that restricted area.

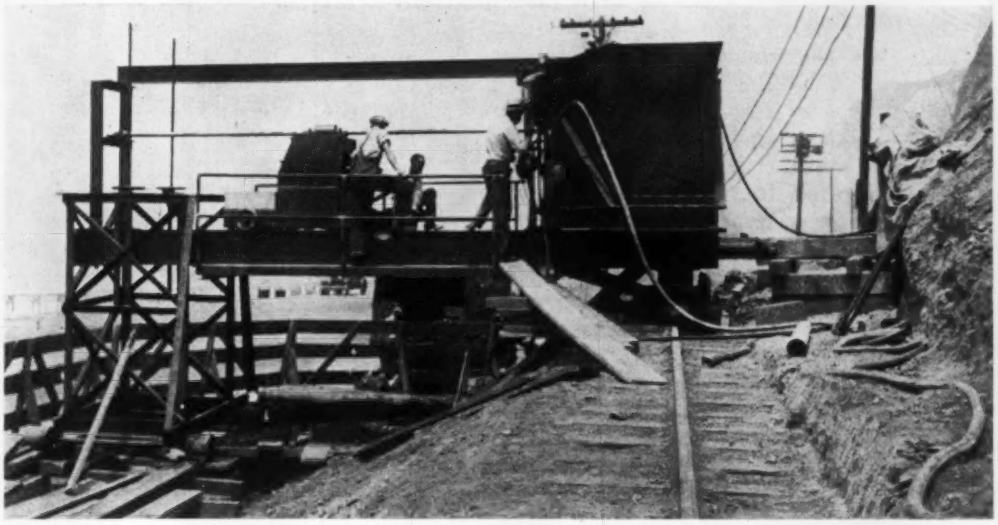
As the tunnels themselves exposed only a relatively small amount of the stratum, additional openings through the clay had to be provided so that drying could be accomplished more effectively. This was done by drilling a series of holes in the face of the bluff extending through the clay stratum and connecting with the main tunnel. The length of the holes ranges from a little less than 100 ft to about 130 ft. Along the face of the bluff, the clay layer is from 3 to 9 ft above the level of the highway and there is little room between the edge of the highway and the toe of the bluff. As the clay is only a few feet in thickness, the holes had to be drilled at a fairly uniform grade, and had to intercept the tunnel, which is only about 6 ft in height. Consequently the accurate drilling of the holes constituted a problem of considerable difficulty. Since no suitable standard equipment could be found, a special machine was built for the purpose. A total of approximately 2,000 ft of holes 10-in. in diameter was drilled.

At the front end, the drilling machine was supported on a rail along which it was moved on wheels, and at the rear end it was hung at any desired elevation in a cage, which was also mounted on wheels. Within the bed frame two hydraulic jacks were provided and were used to force heavy casing into the hole; two other jacks were used to pull the casing from the hole. Within the casing there was a conveyor screw, to the end of which a special bit was attached for cutting out the material directly in front of the casing. The conveyor and the pump actuating the jacks were driven by the same 50-hp motor.

While the casing was being forced into the hole the thrust of the hydraulic jacks was taken up by two screw jacks extending downwards at an angle and bearing on the firm ground along the edge of the highway. The upward component of these inclined jacks was balanced by water ballast in the large tanks on the front end of the machine. When the casing was being withdrawn from the hole, another pair of screw jacks in the plane of the casing were inserted between the machine and the face of the bluff.

Because of the great likelihood of another slide, it was deemed inadvisable to bring out the cuttings with water. A spinal conveyor was therefore utilized for this purpose. The clay in many places was so damp and so plastic that supporting spiders could not be used along the conveyor, which had to be kept in place by permitting the edge of the conveyor flights to bear on the casing. This casing was removed after drilling so that the air passing through the hole would be in contact with the surface of the clay.

For the first 80 ft, the pressure required to force the casing into the hole was comparatively low, but beyond this point it increased rapidly until as much as 600 lb per sq in. was needed on the two 7-in. hydraulic jacks used. This is roughly equivalent to a thrust of 23 tons. It was found that the clay tended to squeeze around the casing and to grip it tightly, although the joints of the casing were perfectly flush. Consequently, the procedure was to drill and advance the casing about halfway on one day and then to drill the remaining distance and pull the casing back to the halfway point on the following day. The crews did not find this difficult to do after they became accustomed to the operation of the machine. Towards the end of the job as much as 80 ft of advance and an equal amount of withdrawal of the casing was accomplished in an 8-hr shift.



Machine Drilling 10-In. Holes in Clay Stratum
Excavated Material Removed by Spiral Conveyor

PRESENT CONDITION OF THE STRATA

Except for a couple of weeks following a fire which destroyed some of the timbers near the portal, the blower has been in continuous operation since July 1933. The furnace was started in August 1933 and has been stopped only when for some reason men had to work in the tunnels.

The humidity of the outside air, which is generally high, has been determined at intervals, and similarly, the moisture content of the air passing out of the various tunnels and drill holes. To the moisture content of the air being sucked in by the blower there must of course be added the water content in the products of combustion of the natural gas fuel. The difference between the total input of moisture and the total output cannot be determined with precision, but in general measurements have indicated that during the first six months about 3,000 lb of water was evaporated each day from the material around the tunnels and drill holes. Subsequently, the rate of removal has become less as the clay has become drier.

Along the tunnels the appearance of the clay has changed markedly. In many places a hard shell has been formed; in others, where the material was somewhat sandy, flaking has occurred. Around the drill holes a crust almost as hard as brick has generally been formed. Samples taken in the sides of the tunnels indicate an appreciable reduction in the moisture content, most of the samples now showing a loss of 15 per cent in weight or less on complete drying as compared to that of more than 20 per cent in substantially all samples taken before drying operations were started.

Originally the temperature in the tunnels was about 72 deg, which is somewhat higher than the mean in that locality, indicating the generation of heat by the movement of the slip. After the blower was started and before the furnace was put in operation, the temperature dropped to approximately the mean temperature of the outside air. After the furnace was started the temperature at the various measuring stations has steadily increased until now it is approximately 110 deg with the furnace shut off at the time of measurement. The entire mass around the tunnels and drill holes has become heated to a depth of at least 6 ft, since samples taken at this distance from the sides have been noticeably warm.

At the present time the heat of the mass around the tunnels and drill holes is sufficient to raise the temperature of the air from the blower without the operation of the furnace for more than half the time. The temperature of the air is of course not as great as when the furnace is in operation, but this lowering of temperature is offset to some degree by the absence of water vapor produced by the burning gas. Some such procedure of intermittent operation will probably be followed, as it is intended that this process of drying out the clay stratum, on which landslips did occur and on which others might occur, will be continued for a considerable time.

The question naturally arises as to whether the cost of the continued operation of the blower and furnace could not be obviated by the construction of some retaining wall or other works to prevent a slide. However, the capitalized cost of operation of the blower and furnace would not go far towards paying for the construction of any permanent structure capable of affording the same resistance to outward movement that evidently is being secured by the increased frictional resistance along the slickensides in the clay. Furthermore, it appears that water is being extracted from the clay much faster than it is accumulating, so that in due course only occasional operation of the blower and furnace will be necessary to maintain the condition desired.

It is recognized that the drying out of a stratum by this process to prevent landslips is an experiment, and that only time can determine whether it will be entirely successful, although at present it appears to be so. The cost of operation is looked upon as insurance against a repetition of the slides of 1932, which would destroy the value of property otherwise very desirable.

Developing the Delaware River

Résumé of the War Department's Report on the Utilization of Its Waters

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Major, Corps of Engineers, U. S. Army, Providence, R.I.

After a four-year study by the War Department, initiated pursuant to the River and Harbor Act of 1927, the Corps of Engineers has recently completed its study on the control and use of the waters of the Delaware River. Because of Major Young's intimate connection with this study, he was especially qualified to prepare this résumé, which he presented before the Philadelphia Section of the Society on March 21, 1934. Hydro-electric power and domestic water supply are the most important future uses. Both head and storage are available for power, and additional domestic water supply will probably be needed in the not far distant future by Philadelphia, northeastern New Jersey, and New York City. Inasmuch as the Delaware River is an interstate stream, now being developed by conflicting interests, the report recommends that major water supply and power developments should be coordinated, supervised, or controlled by an interstate agency.

In 1927 Congress directed the Corps of Engineers of the U. S. Army to submit a comprehensive series of reports covering the more important navigable streams of the United States. These reports analyze the needs and the possibilities for improvements in the interests of navigation, hydro-electric development, flood control, and irrigation.

At the present time consideration is being given at Washington to a national program for watershed development. The series of reports prepared by the Corps of Engineers is the most important source of information for the development of these plans. The Delaware River Report presents the only comprehensive study yet made for the improvement of the water resources of the Delware Valley. When it is considered that the great industrial area comprising New York City, northern New Jersey, and eastern Pennsylvania, including Philadelphia, with a population of about 13,500,000 in 1930, can draw on the Delaware for hydro-electric power and municipal water, the importance of the river is evident.

SCOPE AND SOURCES OF DATA

Work on the Delaware River Report was started in 1928 and completed in 1932. The report was prepared by the District Engineer, U. S. Army, at Philadelphia, Pa. Its scope was extended beyond the four subjects indicated in the congressional authorization to include the subject of water supply because it was felt that the domestic use of water by the large populations of New York City, northeastern New Jersey, and Philadelphia was the most important consideration in connection with the study, and that a report omitting this subject would be inadequate.

This report had no direct connection with the Delaware Diversion Case before the U. S. Supreme Court to which the states of New York, New Jersey, and Pennsylvania were parties, although the District Engineer at Philadelphia was called as a witness before the Master in the case and presented data gathered in connection with studies for the report. The Supreme Court decree in this case influenced greatly the grouping of projects because every effort was made to conform to the legal viewpoint set forth and to meet the requirements stated.

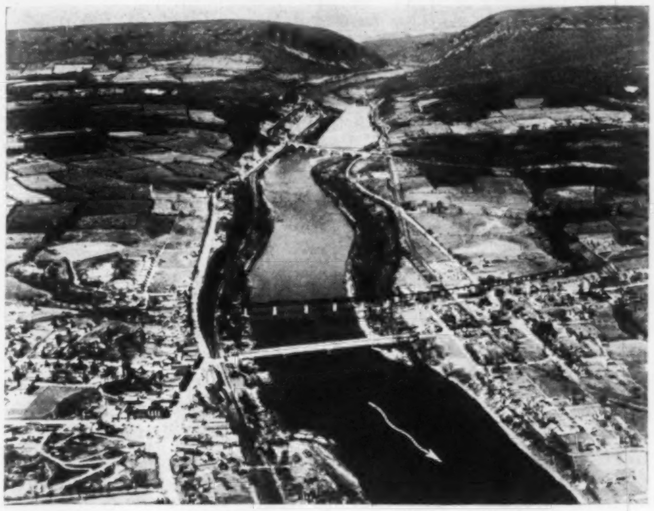
The report on the Delaware River and a similar one on the Lehigh watershed deal with the Delaware watershed above Trenton. The need for flood control measures was considered and was found to be largely limited to the lower Lehigh Valley. It appears that the danger of damage from this cause is not sufficient to warrant the necessary expenditures for control measures. However, in connection with the proposed power and water supply projects, a substantial degree of flood protection

Table I. Storage Sites on the Delaware and Lehigh Rivers (Numbers Refer to Locations Shown on the Map, Fig. 1)

Location	Spillway Elevation Ft	Maximum Storage Depth Ft	Reservoir Capacity in Acre-Feet: Total	Reservoir Capacity in Acre-Feet: Effective
On Delaware River:				
(1) Delhi	1,560	160	49,200	41,200
(2) Cannonsville	1,136	136	295,000	275,500
(3) Downsville	1,250	155	324,000	258,000
(4) East Branch	1,080	83	65,000	53,000
(5) Lew Beach	1,660	118	28,600	27,400
(6) Livingston Manor	1,560	120	31,500	28,800
(7) Hancock	955	65	27,500	20,000
(8) Hankins	822	22	3,800	1,600
(9) Callicoon	850	95	11,900	10,100
(10) Cochecton	743	25	7,250	4,250
(11) Narrowsburg	715	40	13,300	8,500
(12) Prompton	1,180	70	18,000	15,900
(13) Honesdale	1,060	80	32,600	23,600
(14) Wallenpaupack	1,190			160,000
(15) Barryville	630	70	34,000	19,000
(16) Shohola Falls	1,190	44	28,600	22,800
(17) Cold Spring	900	70	5,050	4,200
(18) Mongaup	520	60	12,750	5,000
(19*a*) Toronto	1,220			27,600
(19*b*) Swinging Bridge	1,070			36,800
(19*c*) Mongaup Falls	934			1,600
(19*d*) Rio	815			14,900
(19*e*) Delaware	630			2,300
(20) Claryville	1,645	193	71,000	69,000
On Delaware River (cont.):				
(21) Woodbourne	1,240	65	23,400	21,300
(22) Denton Falls	1,070	125	27,500	24,500
(23) Oakland Valley	710	85	4,500	3,200
(24) Godeffroy	600	135	592,700	588,900
(25) Tocks Island	420	120	657,600	537,900
(26) Belvidere	280	48	38,900	16,900
(27) Chestnut Hill	200	33	10,900	3,900
On Lehigh River:				
(28) Bear Creek	1,280	55	2,200	1,500
(29) Tobyhanna	1,530	140	87,000	80,500
(30) No. 1 (Mud Run)	1,500	155	28,500	24,500
(31) No. 2 (Stony Creek)	1,500	120	6,650	6,000
(32) No. 3 (Bear Creek)	1,500	140	12,560	12,500
For Philadelphia Water Supply:				
(33) Tohickon	400	95	60,690	57,890
(34) Warrington	300	79	98,700	73,200
For Northeastern New Jersey Water Supply:				
(35) Wanaque	300.3		8,900	8,900
(36) Boonton	305			
(37) Morristown	330	40	61,000	51,000

can be afforded. It further appears that improvements for the benefit of navigation above Trenton cannot be justified on economic grounds and that the use of Delaware water for irrigation is unimportant.

Data had been accumulating in the files of the District Engineer Office for many years. The actual study extended over a period of approximately four years and involved the procurement and analysis of all available rainfall and run-off data and the study of surveys of possible reservoir sites for power and water supply. The large amount of existing survey data was supplemented by a limited amount of field work.



THE DELAWARE RIVER, LOOKING TOWARD THE DELAWARE WATER GAP

The Delaware and Lehigh reports present data in some detail for 34 power and storage sites in connection with the hydro-electric and water supply developments considered. Individually, the sites are not worthy of development, but in groups providing extensive storage and regulation they become economically justified. Many additional sites were studied but not considered of sufficient merit to be included in the report. Three gravity water supply conduits were considered, as follows: from Barryville to Godeffroy to New York City, from Tocks Island to northeastern New Jersey, and from Tocks Island to Philadelphia. The principal storage and power sites, and the proposed conduits are to be found in Fig. 1, and their capacities and characteristics are given in Table I.

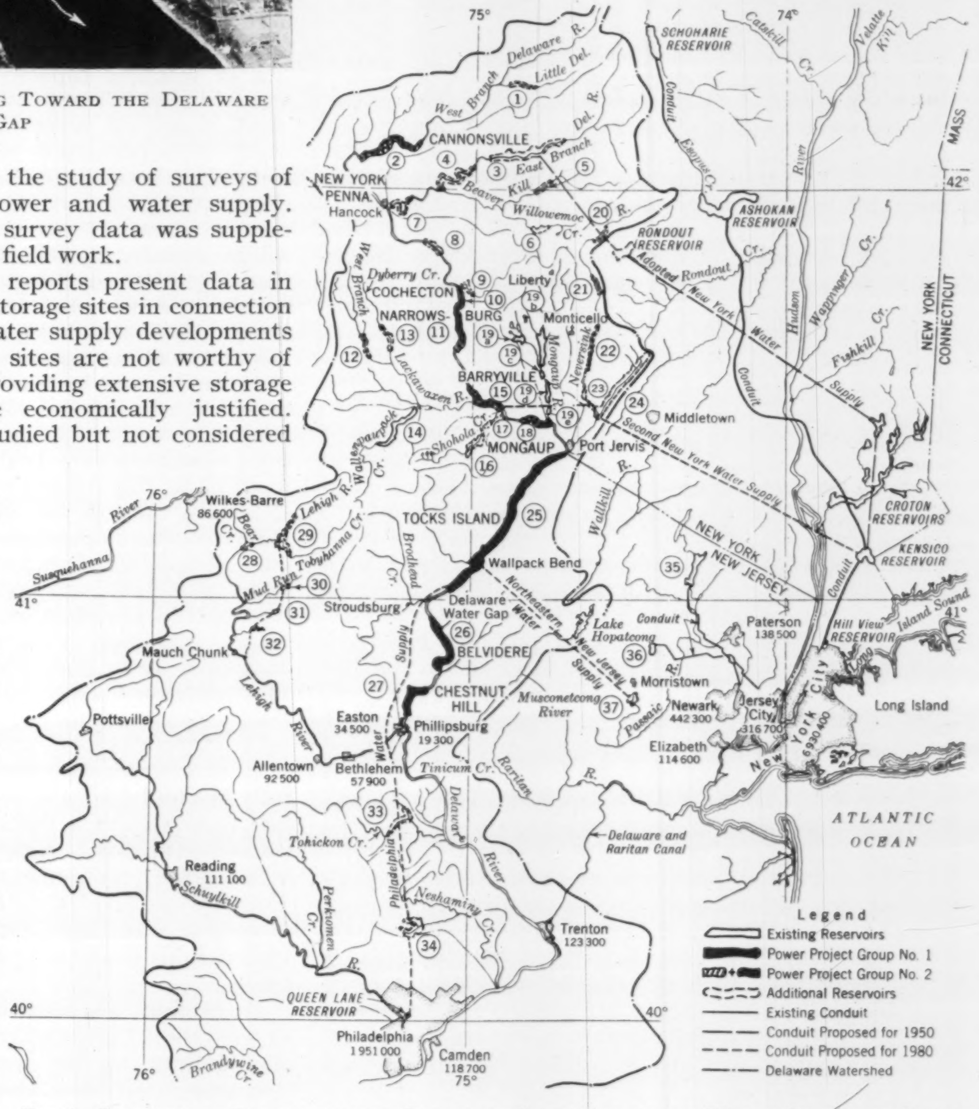
Borings were not made at the various dam sites considered. Complete and satisfactory borings at so many places would have cost a great deal—a much larger sum than had been provided for the entire survey. Foundation conditions along the Delaware River above Belvidere are difficult because the valleys are deep, consisting of rock gorges partially filled with glacial deposits. Many borings and a heavy expenditure would be needed to explore a site adequately. It would require the type of study that should be made by the engineers actually engaged in constructing the dams. It was felt that in a general sense underground conditions were known, and that borings at the various sites considered should not be made as a part of the preliminary study. The estimates are believed to include an adequate margin to cover the study of foundation conditions and the uncertainties of difficult sites.

All estimates in the report are based on 1930 price levels. A special effort was made to make these estimates representative of costs in that year.

The data assembled in the course of this study have been indexed and are available in the Engineer Office at Philadelphia. Interested engineers are invited to avail themselves of the compilation.

REACH FROM BRODHEAD CREEK TO PORT JERVIS IMPORTANT

In any study of the upper Delaware, the importance of the reach from Brodhead Creek, immediately above the Water Gap, to Port Jervis, must be emphasized. This is the only section of the main river valley that does not contain one or more railroads, is free from large


75°
74°
76°
42°
41°
40°
West Branch Delaware R.
Little Del.
East Branch Del. R.
Beaver Kill
Willowemoc Cr.
CANNONSVILLE
SCHOHARIE RESERVOIR
Catskill Cr.
Velatte Kill
Conduit
Esopus Cr.
ASHOKAN RESERVOIR
RONDOUT RESERVOIR
Adopted Rondout New York Water Supply
Hudson River
Wappinger Cr.
Fishkill Cr.
MASS
NEW YORK
CONNECTICUT
NEW YORK
PENNA.
Hancock
West Branch
Dyberry Cr.
COCHECTON
NARROWS-BURG
Liberty
Monticello
Mongaup R.
Neversink R.
Lackawaxen R.
BARRYVILLE
MONGAUP
Port Jervis
Middletown
Second New York Water Supply
Wallenpaupack Cr.
Shohola Cr.
Wallkill R.
TOCKS ISLAND
Wallpack Bend
NEW YORK
NEW JERSEY
CROTON RESERVOIRS
KENSICO RESERVOIR
Hill View RESERVOIR
Long Island Sound
Wilkes-Barre 86 600
Susquehanna River
Bear Cr.
Lehigh R.
Tobyhanna Cr.
Mud Run
Brodhead Cr.
Stroudsburg
Delaware Water Gap
Northeastern New Jersey Water Supply
Lake Hopatcong
Conduit
Paterson 138 500
Mauch Chunk
BELVIDERE
Morristown
CHESTNUT HILL
Musconetcong River
Passaic R.
Newark 442 300
Jersey City 316 700
New York City 6 930 400
Long Island
Pottsville
Lehigh River
Easton 34 500
Phillipsburg 19 300
Water Supply
Allentown 92 500
Bethlehem 57 900
Tinicum Cr.
Raritan R.
Elizabeth 114 600
Delaware River
ATLANTIC OCEAN
Tohickon Cr.
Delaware and Raritan Canal
Reading 111 100
Schuylkill R.
Perkiomen Cr.
Philadelphia
Neshaminy Cr.
Trenton 123 300
QUEEN LANE RESERVOIR
Philadelphia 1 951 000
Camden 118 700
Brandywine Cr.
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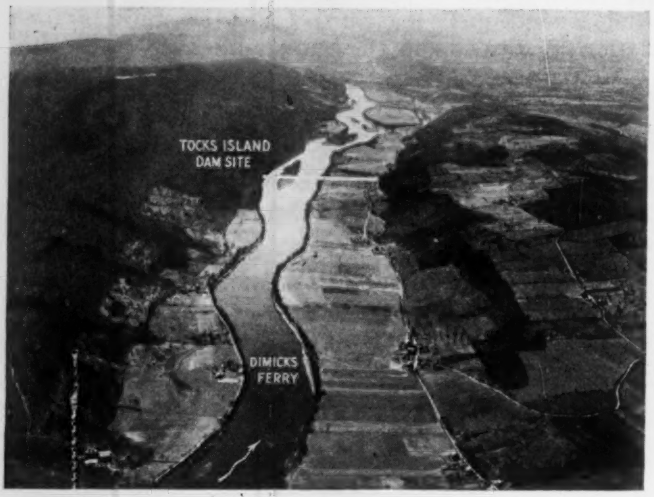
Legend
Existing Reservoirs
Power Project Group No. 1
Power Project Group No. 2
Additional Reservoirs
Existing Conduit
Conduit Proposed for 1950
Conduit Proposed for 1980
Delaware Watershed

FIG. 1. GROUPING OF PROJECTS ON THE DELAWARE RIVER SYSTEM FOR WATER SUPPLY AND POWER

Data Regarding These Sites Are Given in Table I towns, and consequently can be developed with a high dam and reservoir of large capacity. There appears to be an economical site for a dam at Tocks Island, 5.5 miles above the Water Gap. Backwater would extend to Port Jervis, which city would limit the overflow elevation to 420 ft above mean sea level. The dam would rise about 120 ft above the present stream bed, and the average operating head for power development would be about 100 ft. This site is of the utmost importance because of the large effective reservoir capacity, 537,900 acre-ft, and because water can be supplied from it by gravity to the cities of northeastern New Jersey and to Philadelphia. In case subsurface conditions should be found unduly difficult, there is an alternative site at Wallpack Bend, 8.5 miles upstream. A dam at the latter location would be less costly, but the effective storage would be much less, and the Tocks Island site is preferable.



SITE OF TOCKS ISLAND DAM
5.5 Miles Above the Water Gap

INTERSTATE CHARACTER OF THE DELAWARE RIVER

A compact between Pennsylvania and New Jersey dating from 1783 prevents the construction of dams in the main channel of the Delaware River below Port Jervis except with the legislative approval of both states. Attempts made in 1925 and 1926 to negotiate tri-state compacts between New York, Pennsylvania, and New Jersey to provide for the utilization of Delaware water did not receive legislative approval in Pennsylvania and New Jersey. New York City now proposes to divert water from tributary watersheds of the Delaware within the State of New York under the provisions of the U. S. Supreme Court decree of May 4, 1931. It is felt that the interstate character of the Delaware has retarded the development of the stream and may have prevented the construction of power dams in the main river valley.

POWER DEVELOPMENT

A substantial start has been made in power development on the Delaware River. Wallenpaupack Creek has an effective developed storage capacity of 160,000 acre-ft, and the four projects on the Mongaup River have a combined effective capacity of 80,000 acre-ft. In 1930 these improvements maintained the minimum flow at Trenton at approximately 1,800 cu ft per sec, whereas in previous years of extreme drought it had fallen below 1,300 cu ft per sec at this point.

Three sites between Port Jervis and Easton—Tocks Island, Belvidere, and Chestnut Hill—constitute the logical initial power development and together form Power Project Group No. 1. With the large storage provided by a reservoir at Tocks Island, Belvidere and Chestnut Hill become highly profitable power sites. The controlling data are given in Table II.

Power Project Group No. 2 adds five sites, Mongaup, Barryville, Narrowsburg, and Cochecton between Port Jervis and Hancock, and Cannonsville above Hancock, to Group No. 1. It includes the largest, best, and most profitable of the power sites. That at Cannonsville is the outstanding addition, with an effective storage capacity of 275,000 acre-ft available through a total average operating head of 470 ft. Table II also contains important estimates in connection with this group.

TABLE II. CAPACITY, COST, AND INCOME OF POWER PROJECT GROUPS

	Group No. 1	Group No. 2
Effective reservoir capacity, in acre-ft	558,700	870,950
Capacity, in kilowatts	176,400	326,600
Energy produced, in kilowatthours	689,211,000	1,148,437,000
Cost, in dollars	29,623,700	46,754,600
Cost of power per kilowatthour, in mills	4.65	4.45
Ratio of income to total cost, in percentage	13.0	14.5

Disregarding the temporary condition due to the depression, a large and growing market exists for hydroelectric power on a peak basis in New York City and northeastern New Jersey. Operation on a 25 per cent load factor is assumed in the estimates. The computation of installed capacity is based on this load factor and assumes full utilization of the primary flow at the plant under the maximum operating head. Primary power on a peak basis should be worth approximately $0.008 per kwhr. To be attractive to private capital, the project groups should show a ratio of income to total cost of about 11 per cent. Assuming that financing will be done by a public agency at a low rate of interest, then interest, amortization, and operating costs would be about 8 per cent of the capital investment. The possibilities for profit with either private or public development of both groups are apparent.

A third group of sites for power development is discussed in the report. This group adds 12 projects of relatively small capacity located on tributaries entering the Delaware above Port Jervis, and as a whole is not justified by the amount of power that can be developed, although some of the individual projects have merit. However, the storage provided by the group as a whole may be required to maintain an adequate regulated flow and satisfactory sanitary and salinity conditions in the river should Philadelphia, the cities of northeastern New Jersey, and New York City all draw on it for their municipal water and should their requirements increase substantially.

IMPORTANCE AS A SOURCE OF DOMESTIC WATER

In connection with the study of water supply, it was fully realized that the report dealt with a subject that had received much thought and had been a matter of some controversy. New York City had adopted a project for the procurement of water from the Delaware. The representatives of Pennsylvania in the Delaware Diversion Case had stated the intention of supplying Philadelphia from the upper Delaware in the not distant future.

No attempt was made in the report to state how, when, or why these cities or those of northeastern New Jersey should draw on the watershed for municipal supply. Assuming that they do elect to use water from the upper Delaware River, the report considers the problem presented and the effect of such use, in particular on the development of hydro-electric power. The probable needs of the communities up to and including 1980 were studied, and these needs were used as a basis for investigating the projects here described briefly.

According to New York City's adopted project 440 mgd are to be taken from the high watersheds of the East Branch and the Neversink. The drainage areas are largely uninhabited and the water should be of high quality. It is to reach the city through gravity conduits.

In view of the large quantities of water required by Philadelphia and the cities of northeastern New Jersey, it is apparent that great economies can be secured by drawing from the Tocks Island site through gravity conduits of large capacity as compared with utilization of high and largely uninhabited watersheds. Aside from the problem of exposure to human contamination, it appears that water from a reservoir at Tocks Island would be soft and of excellent quality. It would require filtration and chlorination—at least facilities for such treatment would have to be always in readiness.

The supply for northeastern New Jersey, as considered in the report, should meet requirements up to 1980. High-pressure water would have to be pumped; low and intermediate pressures would be supplied by gravity. The proposed construction would provide a maximum capacity of 388 mgd to be drawn on through a gravity conduit 33 miles long from the Tocks Island Reservoir to a regulating reservoir at Morristown. The estimated costs are:

Conduit from Tocks Island to Morristown	$28,766,000
Reservoir at Morristown	5,642,000
Share of additional clearing for Tocks Island Reservoir	1,250,000
Total	$35,658,000

The Philadelphia Supply should also meet requirements to 1980. It would provide 639 mgd by gravity conduit from the Tocks Island Reservoir to a regulating reservoir at Warrington and thence to Queen Lane, a total distance of 76.4 miles. A small and economical impounding reservoir at Tohickon would add to the supply en route. The estimated costs are:

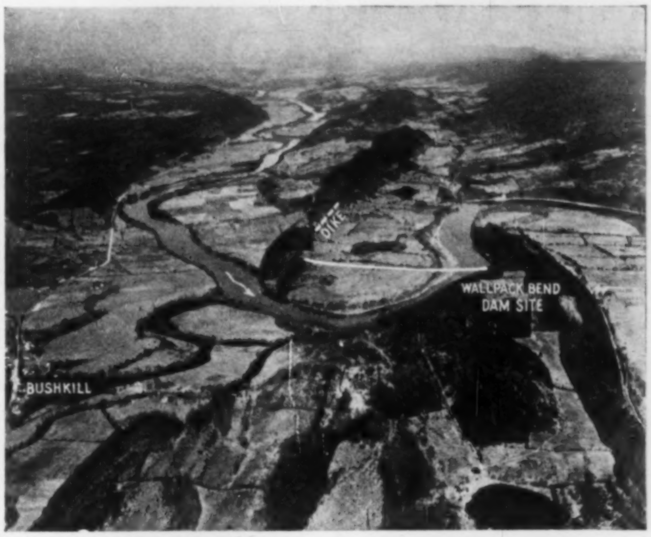
Conduits from Tocks Island to Philadelphia	$76,473,000
Reservoirs at Warrington and Tohickon	13,827,000
Share of additional clearing for Tocks Island Reservoir	1,250,000
Total	$91,550,000

Supplies for northeastern New Jersey and Philadelphia could be developed separately or in combination. There are a large number of possible combinations with power projects which by the sale of power would contribute substantially to the amortization cost. In Table III is presented probably the most representative and advantageous combination, consisting of the northeastern New Jersey and Philadelphia water supplies and Power Project Group No. 2.

Table III. Combination Project for Philadelphia and Northeastern New Jersey Water Supplies Together with Power Project Group No. 2

Water supplied, in million gallons daily:	
Northeastern New Jersey	388
Philadelphia	639
Capacity, in kilowatts	243,900
Energy produced, in kilowatthours	928,178,000
Gross cost, in dollars	169,744,400
Value of power capitalized at 8 per cent, in dollars	61,525,000
Net cost, in dollars:	
Northeastern New Jersey Water Supply	27,796,700
Philadelphia Water Supply	80,422,700
Cost of water per million gallons daily, in dollars:	
Northeastern New Jersey Water Supply	71,500
Philadelphia Water Supply	126,000

This combination represents a step in the development of the watershed which is many years distant.



Site of Wallpack Bend Dam, 8.5 Miles Above Tocks Island

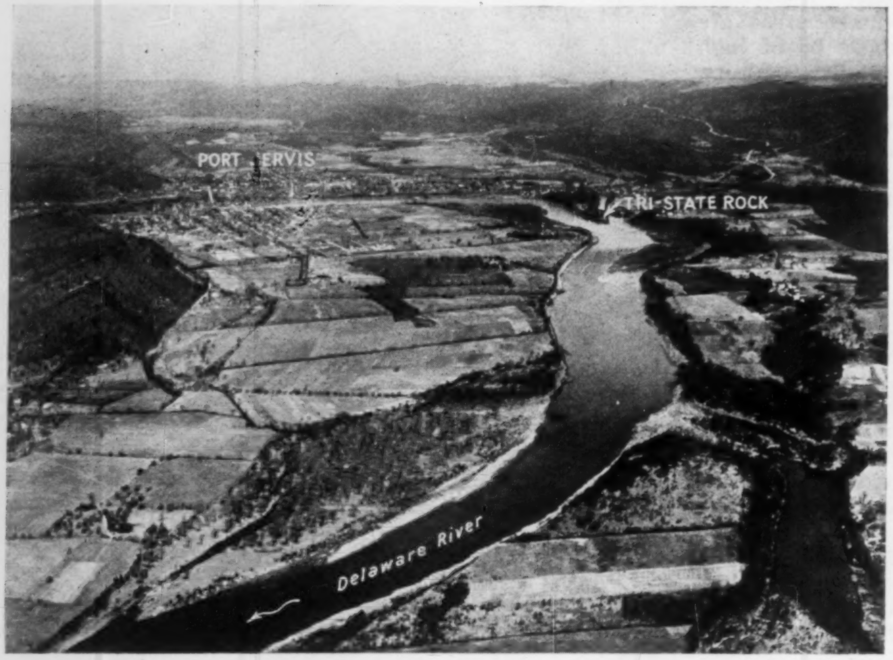
There would be earlier periods during which the quantities of water withdrawn for municipal use would be less and the cost of the water per unit of volume correspondingly greater. It should be noted that according to this allocation of costs, power development is carrying the entire expense of all power reservoirs and is assisting in meeting the cost of water supply conduits and reservoirs.

Requirements up to about 1950 should be met by the adopted New York City development. Since additional withdrawals may be needed for the city after that date, the Second New York Water Supply Project has been prepared to meet its needs up to 1980. If this project should be carried out, it appears that most of the smaller storage projects in addition to those pertaining to Power Project Group No. 2 would be required in order to maintain satisfactory non-saline and sanitary conditions in the vicinity of Philadelphia, Chester, and Wilmington. The second New York project is complicated; the estimates are large; and it is a long way off in point of time. Details cannot be given in this brief résumé.

SALINITY AND SANITARY CONDITIONS IN THE ESTUARY

Much study has been given to the effect upon salinity and sanitary conditions in the vicinity of Philadelphia, Chester, and Wilmington of diversions to New York City and northeastern New Jersey. The Philadelphia Engineer District has been studying the problem since 1929, and the Commonwealth of Pennsylvania since 1931. Private interests have also worked on it.

The regulating effect of the Wallenpaupack Creek and the Mongaup River group is already apparent. The projects under consideration would eliminate periods of extremely low flow. In the New York Diversion Case, the Supreme Court determined that releases from dams constructed in connection with the adopted New York diversion shall be made whenever the river flow falls below 0.50 cu ft per sec per sq mile of drainage area above Port Jervis and Trenton, or 1,555 cu ft per sec at Port Jervis and 3,400 cu ft per sec at Trenton. Releases are limited in quantity and frequently will not maintain the flow at Trenton at 0.50 cu ft per sec per sq mile. In the Delaware River Report the measure of regulation established by the Supreme Court was accepted as desirable, and in the case of diversions from the watershed to New York City and northeastern New Jersey, storage was provided to supply it. The combination project described would meet this requirement.



THE DELAWARE RIVER AT PORT JERVIS, N.Y.

Accepting as a fact the theory that domestic supply is the highest use of water, it follows that industrial interests should not be permitted to hinder the development of the proposed water supplies, even though regulation and diversion may prove somewhat detrimental to them. The power projects considered would modify the flow, storing water in time of flood and increasing the low-water flow but would not divert water from the river. It appears that the degree of salinity and sanitary conditions in the estuary should be observed continuously so that any changes resulting from the construction of the proposed projects may be established.

Assuming diversion to New York City and northeastern New Jersey, the water withdrawn from the watershed will be only a minor percentage of the total run-off. More specifically, if New York City and New Jersey were drawing respectively 440 and 388 mgd daily, these diversions would total 10 and 6.7 per cent of the mean annual run-off at Trenton and the capes, respectively. It is believed that experience alone can determine the amount of storage and regulation required; also that adequate storage within the capacity of the reservoir systems studied, and properly controlled releases should prevent an adverse effect on the salinity and sanitary conditions in the vicinity of Philadelphia, Chester, and Wilmington.

CONCLUSIONS

Technically, the projects proposed in the Delaware River Report are closely interrelated, and at the same time are subject to the divergent business and political interests of three states and many cities. It appears that future major water-supply and power developments should be subject to the supervision, coordination, or control of a single interstate agency, which might act in a supervisory capacity or might actually carry out the proposed developments. The interstate character of the river, the possibility of ill-advised and partial developments, the complex problem presented with respect to salinity and sanitary conditions in the estuary, all indicate the necessity for a controlling agency.

The report contains the following conclusions:

a) The improvement of the Delaware River for navigation above Trenton is not economically justifiable under present or prospective conditions.

b) The Delaware River is of great value as a source of water supply for New York City, and probably at a future date for the cities of northeastern New Jersey and Philadelphia.

c) Existing and potential hydro-electric values are substantial, and power developments may be combined advantageously with the storage and regulation necessary for municipal water supply projects.

d) Future major water supply and power developments within the Delaware watershed should be coordinated, supervised, or controlled by an interstate agency.

e) Federal participation in any present or prospective project above Trenton does not appear justifiable.

In connection with the last statement (*e*), that Federal participation in any present or prospective project above Trenton does not appear justifiable, it must be remembered that the report was prepared in 1932, prior to the present Federal public works policy and program. It is conceivable that some of the projects considered in the report might properly find a place within the public works program.

Earl I. Brown, M. Am. Soc. C.E., Colonel, Corps of Engineers, and District Engineer at Philadelphia, was responsible for the preparation of the report. C. S. Jarvis, M. Am. Soc. C.E., Senior Engineer, was in charge of the earlier studies and was largely responsible for the gathering and analysis of the rainfall and run-off data. I was in direct charge of the study during 1930, 1931, and 1932 and prepared the report. V. F. Spring, Assistant Engineer, of the District Office, participated in the study throughout, and is in charge of, and entirely familiar with the records of the survey. Also John P. Hogan, Eugene E. Halmos, and H. Alden Foster, all Members Am. Soc. C.E., of the firm of Parsons, Klapp, Brinckerhoff and Douglas of New York City, participated as consultants and reviewed all the studies and estimates. Mr. Hogan contributed particularly to the report as it now stands in the grouping of power and water supply projects and in connection with the estimates for returns from the sale of power.

In accordance with the usual procedure in the Engineer Department, the report was reviewed and approved by George M. Hoffman, Colonel, Corps of Engineers and Division Engineer at New York, N.Y., by the Board of Engineers for Rivers and Harbors, at Washington, D.C., and by the Chief of Engineers.

The report is now in the hands of the printer and should be available shortly. The aerial photographs accompanying this résumé were taken by the Dallin Aerial Surveys, Inc.

Safe Design of Highway Guard Rails

Laboratory and Field Tests in Georgia Indicate Requirements for Effective Types

By SEARCY B. SLACK
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CONSULTING ENGINEER, DECATUR, VA.

GUARD rails along highways are placed both as a visible warning at danger points and as a positive protection to prevent vehicles from accidentally leaving the roadway at these critical places. A paucity of test data led the State Highway Board of Georgia to make laboratory and field tests on four types of guard rails—wooden rails, steel plate rails, woven wire fabric of various kinds, and wire cables, all supported on wooden posts. In this article, Mr. Slack shows the advantages and disadvantages of these types as revealed by the tests and offers pertinent suggestions regarding their design and construction. He has made a thorough study of the subject, and his conclusions should be generally helpful to highway engineers.

HIGHWAY safety is rightly receiving more and more emphasis, and as part of this phase of the work, the guard rail deserves attention. The purpose of a guard rail along a highway is to protect traffic. To afford this protection the rail must serve as a guide to traffic, as a warning at points of danger, and in case it is struck by a vehicle it must be so designed as to cause the least possible damage to the vehicle while keeping it from leaving the road. The desirable qualities of a rail then are visibility, strength, and cushioning action in case of impact.

From the standpoint of maintenance the guard rail should be easy to repair after an accident; it should be self-straightening following light contacts by vehicles; the parts should be reduced to a minimum; and they should be simple and preferably not useful on the farm.

Reliable information as to the behavior of various types of guard rails under the impact of a vehicle is rather meager. Comparatively few tests have been recorded, and reports on accidents are so lacking in detail and so conflicting that it is difficult to draw conclusions. The observations and results here presented are based on laboratory and field tests of highway guard rails made for the State Highway Board of Georgia.

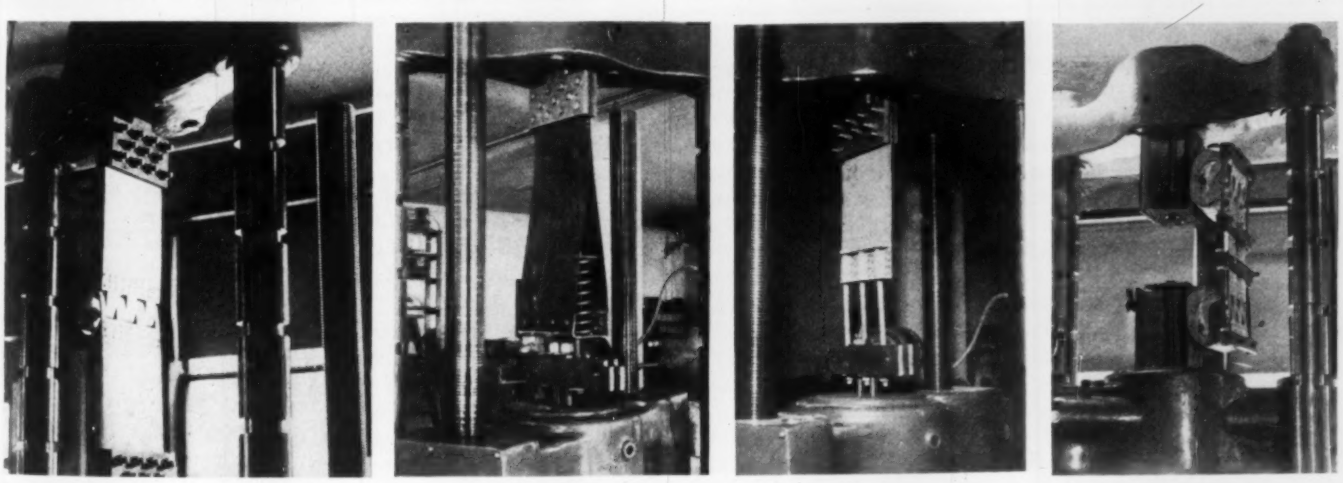
LABORATORY TESTS OF FULL-SIZE ELEMENTS

Since the laboratory testing of guard rails is new, much remains to be done in perfecting the technic and standardizing the methods. The tests used in Georgia give a valuable method of determining the strength of assemblies and balancing the design of the various elements used in making up the rail. They also give interesting comparisons of the strength and elastic properties of various types of guard rails when considered in connection with field service records of types that have proved satisfactory.

In Georgia the tests included a determination of the longitudinal strength and elasticity of the intermediate connections and end assemblies of the rail, a side-pull test to measure the strength of the post connection, and a crushing test to determine the strength and elastic properties of the offset fittings. No reference is made here to the usual tests to ascertain the quality and properties of the materials used in the rails.

For testing the plate-type rail in tension, special grips were made for holding the full plate sections in the testing machine. Details of the grips are shown in Fig. 1. To fasten a section of plate in the grips, the plate was placed between the jaws of the grips and laid flat in the testing machine, which, acting in compression, pressed the ridges into the plate. Holes were then punched in the plate to correspond with those in the grips, and grips and plate were bolted tightly together.

In testing cables, wire tape, or narrow bands of fabric, the sections to be tested were fastened between the regular end connections or fittings, and the bolts for these fittings were placed between the grips in the head and bed of the testing machine. Such procedure gave a test



Intermediate Spring Element — End Assembly with Spring — Bolted End Assembly — Test of Offset Bracket

TESTING VARIOUS ELEMENTS OF PLATE-TYPE GUARD RAILS of the end fittings rather than of the rail section, but this was desirable because in all tests previously made, failure occurred in the fittings. The strength of the individual wires in the tape, fabric, or cable was also tested.

A side-pull test is essentially a test of the offset fittings, or other fastenings, between the rail and the posts. To make this test, a short section of the rail was fastened in the usual way between two offset fittings which were in turn fastened to 8 by 8-in. steel H-sections secured to the head and bed of the testing machine. The H-sections replaced the posts, so that when the head and bed of the machine moved apart the effect was the same as though two posts connected by the guard rail were moved in a similar manner. From this test could be determined the force required to cause a slip of the cable, tape, or strip under the fastening used to connect it to the post, and also the strength and elastic properties of the offset fittings if these were used.

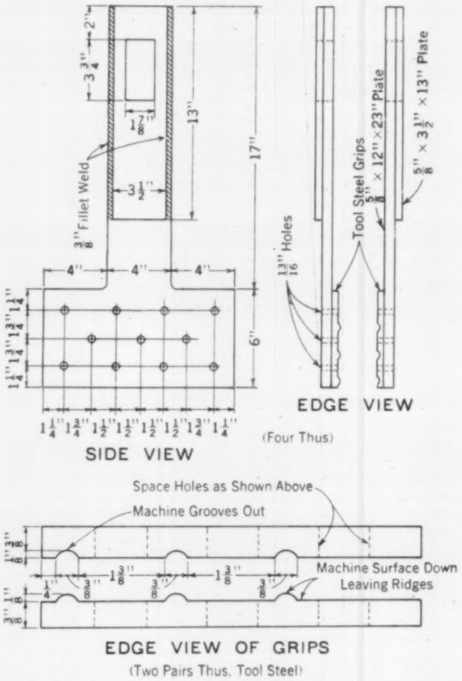


FIG. 1. GRIPS FOR HOLDING SPECIMENS OF GUARD RAILS IN THE TESTING MACHINE

The purpose of the connection between rail and post is to support the rail and hold it in line, to transmit a part of the force of the impact from the rail to the post, and in case the rail should be broken, to localize the break, leaving the other panels of the rail in serviceable condition until repairs can be made. The strength of the connection should be sufficient to develop a large part of the lateral post resistance. The side-pull strengths shown by tests on three types of plate guard rail were from 4,500 to 5,900 lb. Types of rails that are not fastened to the intermediate posts, except as guides, have no side-pull strength.

Crushing tests of the offset fittings were made by placing a specimen between the head and bed of the testing machine and applying a compressive load. The elastic properties as well as the force required to crush the fitting were measured. These tests determined the cushioning effect of the fitting and the distortion to be expected under various loads.

FIELD TESTS UNDER SIMULATED TRAFFIC CONDITIONS

Two sets of field tests were made, the first in 1931, and the second in 1933. The site chosen for these tests was a steep hillside with a level field at its foot near East Point, Ga. A timber runway was built down the hill, and the guard rail to be tested was erected on comparatively level ground across the end of the runway. The runway served to guide the vehicles into the guard rail and determined closely the point of impact and the angle. When the vehicles were given a fast shove with a truck at the top of the runway, they would attain a speed of from 15 to 25 miles per hr at the bottom. After the impact, the vehicles, deflected by the rail, would stop themselves by rolling across level ground.

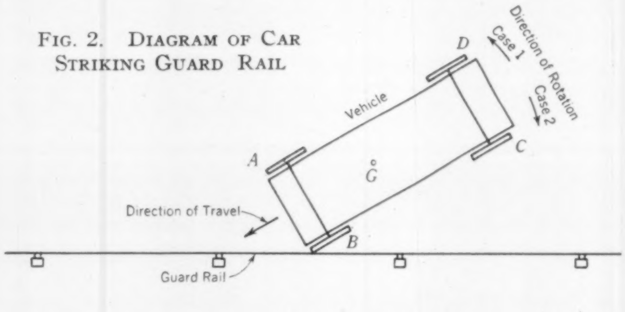
In the first set of tests, the angle between the runway and the guard rail was made either 20 or 40 deg, but in the second set this angle was in all cases 20 deg. Observations of actual traffic impacts of vehicles against guard rails indicate that in 90 per cent or more of the cases, the angle between the direction of travel of the vehicle and the guard rail is less than 20 deg. The tests at an angle of 40 deg, in the first part of the investigation, were intended as strength tests.

In the first set of tests the center line of the runway was midway between two posts of the guard rail. This caused the left end of the bumper on the vehicle to strike the rail from 2 to 5 ft from the post, depending on the angle and the distance between posts. In the second set of tests, the posts were set so that the left end of the bumper would strike midway between two posts. The earth in which the posts were set was a stiff red clay containing from 50 to 60 per cent of sand and gave excellent support to the posts. Five panels of rail were generally set for a test; that is, two end panels and three intermediate panels. The vehicles struck the center panel.

The vehicles used in the tests were passenger cars of medium size, weighing from 3,200 to 3,600 lb; a truck weighing 11,300 lb empty and 14,000 lb loaded; and a passenger bus weighing 10,800 lb empty and 16,300 lb loaded. The passenger cars had bumpers that were considered to be of average construction. The bumper on the truck was made of two 4 by 6-in. oak timbers backed with 6-in. I-beams fastened and braced to the frame and backed by four coiled springs. This bumper was 6 ft long, 12 in. wide, and about 12 in. above the ground. These dimensions cover the usual range of bumpers on passenger cars. The truck was used as a strength test, since high speeds could not readily be secured with the passenger cars.

Measurements were made to determine the position of the post supporting the guard rail before and after each test, and a record was kept of the lateral and longitudinal movement resulting from the impact. In the first set of tests, the speeds of the vehicles at the time of impact were determined by timing the travel over the last 50 ft with a stop watch. The speed was also checked by noting the number of frames of motion picture film required for the vehicle to pass by a row of stakes set 5 ft apart and parallel to the runway. The time interval between the frames was determined by taking a motion picture of a clock with a large second hand. By counting the number of frames required for a given travel distance, the speed of the vehicle could be estimated.

FIG. 2. DIAGRAM OF CAR STRIKING GUARD RAIL



In the second set of tests, the speeds of the vehicles were measured by an electric timing device. Two switches were arranged 25 ft apart at the lower end of the runway so that the wheel of the vehicle in passing closed the first, or upper switch, and opened the lower. The time interval between the closing and opening of the switches was determined from marks made on a paper disc rotated at a known speed on the table of a phonograph. An electromagnet caused a jog in the line on the paper disc when the circuit was closed and another jog when the circuit was opened. The angular distance between these jogs gave the time interval, from which the speed could be computed.

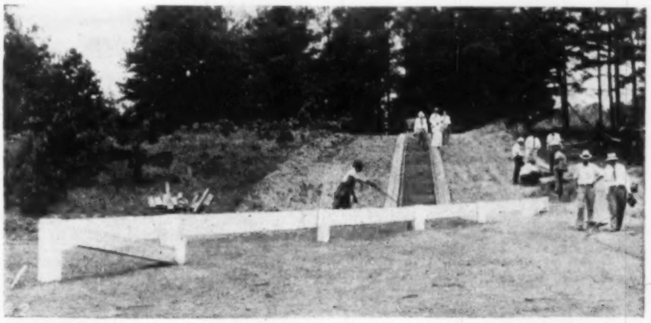
Both motion and still pictures were taken of all tests. Two and sometimes three motion picture cameras were used. The best idea of the behavior of the vehicles and rails can be secured from a study of these motion pictures.

TYPES OF GUARD RAIL TESTED

In all tests the posts were of pine timber, generally 6 by 8 in. in section and 7 ft long, although in a few of the tests round pine posts from 7 to 8 in. in diameter were used. The posts were buried in the stiff sand-clay soil from 3 ft 8 in. to 4 ft 8 in.

Four general types of guard rails were tested: first, wooden rails 4 by 8 in. and 4 by 10 in. in cross section, protected by a 16-gage metal face plate fastened with lag screws, the rails being attached directly to the posts; second, rails of steel plate attached directly to the posts or through offset fastenings of various types; third, steel cables and rods attached directly to the posts or through various types of offset fastenings, with and without ties connecting the cables together between the posts; and fourth, a number of types of woven wire, expanded metal, and steel fabric rails attached to the posts either directly or through spring offset fastenings. The spacing between the posts was in general 10 ft for the wooden guard rails and 16 ft for the other types. In general, the bottom of the guard rails was maintained at a height of from 12 to 15 in. above the ground. The cables, tapes, and narrow strips of wire fabric were spaced 10 in. apart on centers.

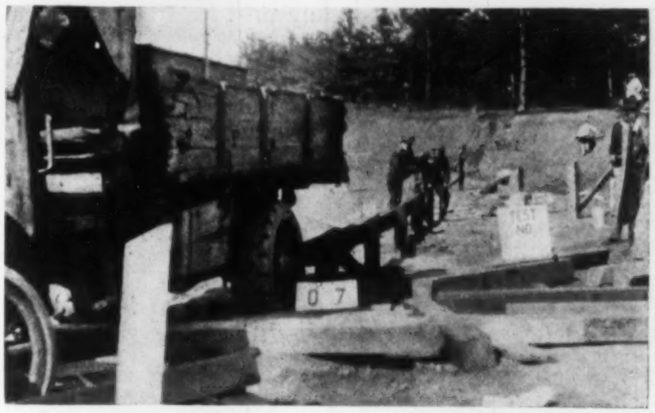
Pine timber posts 6 by 8 in. in section or 7 in. in diameter and 7 ft long, buried not less than 4 ft in the ground, had ample strength to carry the rails tested. Posts should not extend above the rail any farther than is necessary, since the fenders of cars and the bodies of trucks project over the rail and strike the upper part of the post as the vehicle glides along the rail. On the other hand, the bolts which fasten the rail to the posts should not be less than 6 in. below the tops of the posts unless provision is made to strengthen the posts against splitting.



Runway and Guard Rail Set Up for Field Test

The rail should be sufficiently strong to withstand the impact of a vehicle and to transmit it to neighboring posts. Only when several posts act together will a guard rail be sufficiently strong to deflect a vehicle.

Field tests showed that properly designed offset fittings at the posts lessened the damage to both rail and vehicle. They served as cushions between the vehicle and the posts, and their spring action helped the vehicle to pass the posts. Rigid offset fittings with corners or angles damaged both vehicle and rail.



Truck Traveling at 17.8 Mph Deflected by Wooden Guard Rail

Steel Face Plates Added Strength and Safety to Rails

Offset fittings at the posts should be protected from a direct blow from vehicles and, if they carry multiple-element rails, should prevent the different strands from spreading apart. Rails without offset fittings at the posts are often effective, but in case the rail is struck the damage to both rail and vehicle is greater than if cushioning offset fittings are provided. When the rail is fastened directly to the posts, except when it is of wood, round posts give the best results, as they do not form sharp corners in the rail when the vehicle presses it back. If the rail is fastened securely to the intermediate posts these are brought into play and help to localize the strain, so that in case it is broken, only two or three panels are affected. If it is not fastened to the intermediate posts, and if it is broken, the whole stretch of rail, from end post to end post, becomes ineffective until repaired.

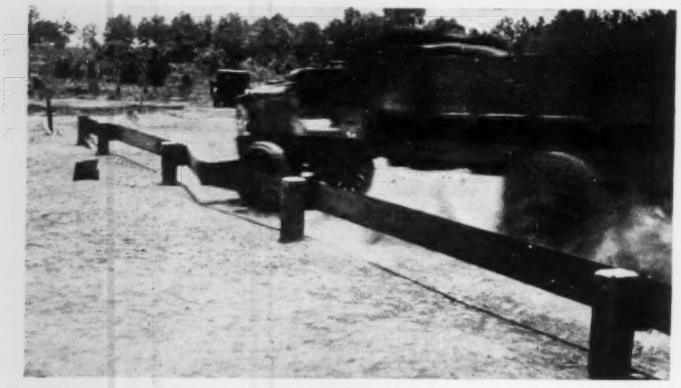
The point of impact is uncertain and the part of the vehicle which will slide against the rail varies widely with the different types of vehicles. These conditions are unavoidable considering the variety of traffic to be expected on the highways. To meet them, a rail should have as much strength as possible, regardless of where it is struck, and should present a wide and smooth sliding surface. Where ample provision is made for fastening the separate strands of a multiple-element rail so that they will act together, the effectiveness of such a rail is greatly improved.

All plate-type rails should have rounded edges. In the 24 tests of this type only one tire or body was cut by the edge of a rail plate, and this was done by a plate with a sheared edge. Steel for guard rails should have toughness as well as strength; brittleness should be avoided by proper composition and heat treatment.

THE DESIGN PROBLEM

An ideal guard rail should be highly visible; in an emergency it should deflect a vehicle back into the road with a minimum of damage to both vehicle and rail; and it should not cause snow drifts. The rail should deflect the vehicle rather than stop or check it. If a guard rail stopped a vehicle going at a high rate of speed within a distance of one or two feet, the damage to the vehicle and passengers would be about the same as if they went over the embankment. Also, such tremendous strength would be required to check suddenly the kinetic energy of a fast-moving vehicle that it would be impractical to build a rail to meet such requirements.

A vehicle consists essentially of a load supported on four wheels. The diameter of the wheels varies from 24 in. to as much as 40 in. for the larger trucks. The great majority of wheels are less than 30 in. in diameter. These are designed to carry the load and to check the speed of the vehicle, and they are securely fastened to the chasis through the springs. The guard rail therefore functions best when it comes in contact with the wheels or bumper and does the least damage to these parts. Any part of the guard rail that strikes above the wheels would not be effective in deflecting the vehicle until it had good bearing against the body, hood, or radiator, all of which, on passenger cars and buses, are rather easily crushed.



STEEL-PLATE GUARD RAIL HIT BY TRUCK TRAVELING AT 21.1 MPH
Importance of Correct Height of Guard Rail Demonstrated

In Fig. 2 is shown a diagram of a vehicle supported on four wheels striking a guard rail. If B strikes the guard rail and the rail tends to check the speed of that corner of the vehicle, then a moment is developed tending to head the vehicle more into the guard rail. The moment will be represented by the amount the corner B is retarded by the rail multiplied by the distance of the center of gravity of the vehicle from the guard rail. The direction of rotation will be as shown by the arrow marked "Case 1." On the other hand, if corner B is not retarded by the guard rail but glides along its face, the reaction from the rail tends to turn the vehicle parallel to the guard rail, and the direction of rotation of the vehicle will be as shown by the arrow for "Case 2." From this it may be concluded that a guard rail which will not entangle the vehicle but will permit it to slide freely along the rail will give the best results.

The support for the guard rail should be resilient, visible, and strong enough to withstand reasonable shock from a vehicle sliding along the rail. Rigid or very heavy posts will cause much more damage to vehicles than lighter, more resilient ones. The distance between posts will vary with the type of rail, but probably too much emphasis has been placed on post spacing. The spacings nsed for the sections of rail tested were 10, 12, and 16 ft. Except in the case of the wooden rail, which depended on beam action, there was no indication that the close spacing gave better results. For a metal guard rail under average highway conditions, a post spacing of 16 ft should prove satisfactory. Where special strength is required, the longitudinal strength of the rail should be increased and the post spacing reduced to 12 ft.

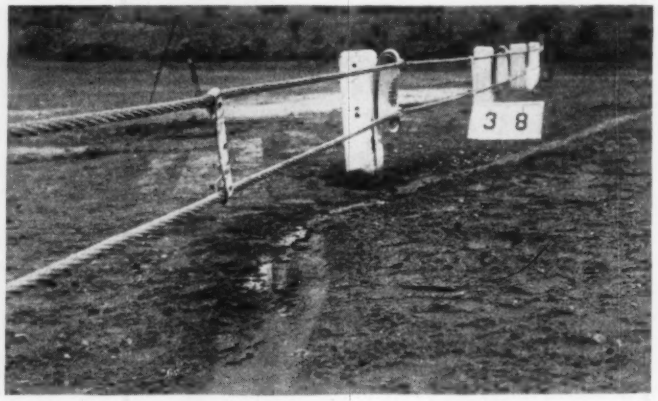
The depth to which the posts are buried and the size of post required to secure proper bearing in the soil should be varied according to the type of the soil. Experience with a particular soil should be the governing consideration rather than any standard rule. In judging posts it must be borne in mind that size for bearing in the soil as well as strength must be taken into account.

DIFFERENT TYPES OF GUARD RAILS

The choice of the best type of guard rail has been the subject of much controversy, but it would appear that several types can be used satisfactorily. If a wooden rail is used, it should consist of fairly heavy timber, not less than 4 by 10 in. in section, and it is improved if the face of the timber is protected by a metal plate fastened by flat or round-headed lag screws or bolts at frequent intervals. The metal plate will prevent the hub or bumper of the vehicle from digging into the timber and will afford a smooth gliding surface. Also, the plate will add somewhat to the strength of the rail and will serve to hold broken ends together. There are numerous accounts of accidents in which a wooden guard rail was broken and forced into the vehicle, sometimes with fatal results. If the wooden rail is bolted to the metal face plate at frequent intervals, it does not seem likely that such an accident would occur.

There are several types of guard rail in which the rail element consists of a steel plate. The points to be considered in this type of rail are: the longitudinal strength of the rail element, including the quality of the metal in the plate; the area of the plate and the type of splices; the fittings for fastening the rail at the posts; and the arrangement for holding the rail plates tight together and at the same time allowing for expansion and contraction due to temperature changes.

The longitudinal strength of the plate rails included in the tests was from 44,700 to 71,000 lb for the intermediate splices and from 38,000 to 70,500 lb for the end connections. The failures that occurred in the rail sections were attributed to causes other than lack of longitudinal strength; hence 45,000 lb may be considered a satisfactory minimum strength for intermediate connections, and 30,000 lb for end-post connections.



STEEL-CABLE RAIL AFTER BEING HIT BY CAR MOVING AT 24 MPH
Deflection of Car Shown by Wheel Tracks

It is necessary that the rail plates be securely fastened to the posts, preferably through a spring element. If the fittings at the posts are rigid, or if the plates are fastened directly to the posts, vehicles striking the rail will be damaged. The tests indicated that the best results are obtained when the rail is attached to the posts by spring offset fittings, which act as a cushion and allow vehicles to slide along the rail and pass the posts without striking them. This reduces the damage both to the vehicle and to the rail.

To be effective, a plate-type rail should be kept constantly tight; a total longitudinal tension of from 2,000 to 3,000 lb should be maintained at all temperatures. This requires a carefully planned arrangement of springs either in the connections or in the rail. With the thin metal generally used, laboratory tests to determine the exact strength of fastenings and connections are essential, as their computed strengths are unreliable.

In the types of rail made up of cables, wire tapes, and narrow strips of steel fabric, the points to be considered are longitudinal strength, height, and spacing of the rail elements, the type of fittings for fastening these elements to the posts, and the method of holding the rail elements tight. The total longitudinal strength of a rail made up of separate elements should be somewhat greater than that of a rail containing only a single element, as it is probable that the separate elements will not be stressed equally. In the field tests, the total longitudinal strength of the rails was found to be from 59,200 to 76,000 lb. This strength was considered satisfactory. A minimum total longitudinal strength of 60,000 lb is suggested—that is, 30,000 lb per element for a two-element rail.

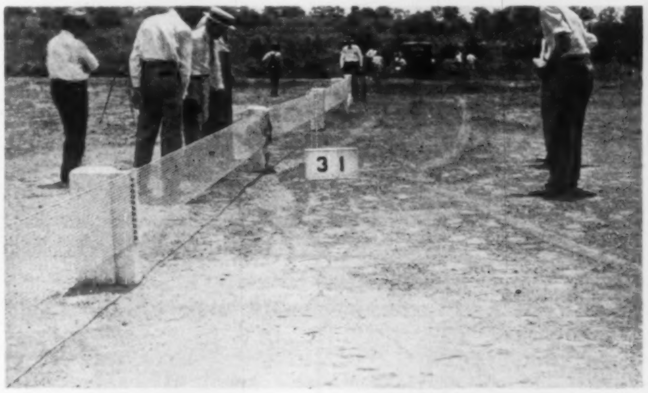
Results of the tests indicate that a height of from 12 to 14 in. above the ground for the bottom cable or tape and a spacing of from 10 to 12 in. between cables or tapes is effective. If the lower cable is too high above the ground, so that the wheel tends to be forced under it or if the cables are spaced too far apart, so that they will not act together, then the effectiveness of this type of rail is greatly reduced. Even with the 10 to 12-in. spacing used in this investigation, the wheel of the vehicle was forced between the cables or tapes in all tests except when the two elements were fastened together by ties midway between the posts.

As regards the fastening of the rail elements to the posts, the same general principles of using offset spring fittings and keeping the rail tight apply to the cable, tape, and strip types as to the plate type previously discussed. It is also important to shield the offset fittings to protect them from a direct blow from a vehicle gliding along the rail. Eye-bolts and J-bolts can hardly be considered as effective fastenings for cables. The fastenings should be sufficiently strong to withstand a pull of not less than 2,000 lb on the rail element before any slip occurs at an intermediate post connection.

A positive tie fastening the cables, tapes, or strips together at the posts and another midway between the posts are apparently necessary to prevent the rail elements from spreading apart when struck by a wheel or bumper. Eleven tests were made on rails consisting of two cable, tape, and strip elements, and in seven of these there was no tie between the rail elements between the posts. In every one of the seven tests the wheel of the vehicle was forced between the two elements. In the other four tests, where the two elements were fastened together, they apparently worked together and were much more effective.

Plate Rail Unattached to Intermediate Posts
Phonographic Speed Recording Device at Right

When the cables or tapes are reasonably well fastened to the intermediate posts, heavy end anchorages are not necessary. In fact, the field tests indicated that such anchorages added little to the effectiveness of the rail. Probably too much attention has been given to the end anchorage and too little to the fastening of the rail to the intermediate posts. Because of their closeness to the point of impact, the intermediate posts must after all exert most of the force required to deflect the vehicle.



Steel Fabric 14 In. Wide After Being Hit by Car at 23 Mph
Car Deflected as Shown by Tracks

LIMITATION OF USE

Essentially, the guard rail is a safety provision, and no discussion of the subject would be complete unless attention were called to the objections to it and to other types of construction which perhaps afford equal safety under many conditions. Among the objections are the facts that the rail adds to the cost of maintenance and prevents the repair of the shoulders by machinery. It also prevents the use of a mowing machine for cutting grass and weeds. Furthermore, it restricts the use of the shoulders for parking or traffic purposes in case of emergency.

In many places the slopes can be flattened or the width of the shoulders increased so as to provide additional safety for traffic without the use of a guard rail. At the usual prices for earth embankment and for most types of guard rails, an embankment 8 ft high could be flattened from a slope of $1^1/_2$ to 1 to that of 3 to 1 somewhat more cheaply than a guard rail could be built. The width of the shoulders on an 8-ft fill could be widened by 7 ft somewhat more cheaply than a guard rail could be provided. On a fill 15 ft high, the shoulders could be widened by 4 ft at about the same cost as a guard rail. Either flattening the slopes or increasing the width of the shoulders is a much safer construction in many cases.

ENGINEERS' NOTEBOOK

From everyday experience engineers gather a store of knowledge on which they depend for growth as individuals and as a profession. This department, designed to contain practical or ingenious suggestions from engineers both young and old, should prove helpful in the solution of many troublesome problems.

Fundamental Theories of Graphical Integration

By M. S. Douglas

Assistant Professor of Civil Engineering, Case School of Applied Science, Cleveland, Ohio

GRAPHICAL calculus plays a useful part in the design of engineering structures. In order to explain some of the fundamental theories involved, for a number of years I have used the following proof in teaching the subject.

Let the accompanying Fig. 1 represent any section. The height y at any point will be some function of the distance from the end, or y equals $f(x)$. The sum of the ordinates multiplied by dx will equal the area and may be expressed as

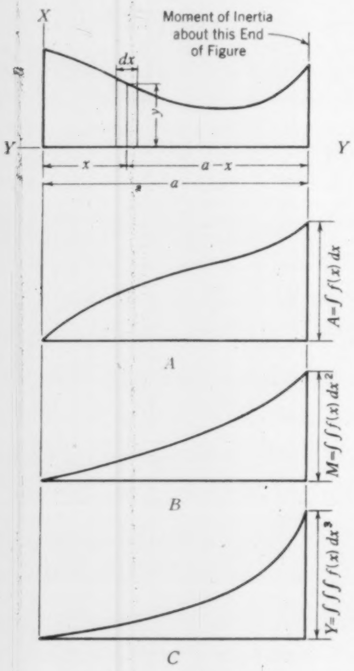

X
Moment of Inertia about this End of Figure
dx
B
y
Y
Y
x
a−x
a
A=∫f(x) dx
A
M=∫∫f(x) dx²
B
Y=∫∫∫f(x) dx³
C

Fig. 1. Derivation of Graphical Method for Determining Moment of Inertia

$$A = \int f(x)\,dx \quad . \; . \; . \; . \; . \; . \; . \; . \quad [1]$$

By starting at the left of Curve A in Fig. 1 and summing the area under the curve, a second curve, B, will be produced. This may be expressed mathematically as

$$M = \int\int f(x)\,dx^2 \quad . \; . \; . \; . \; . \; . \; . \quad [2]$$

This is the expression for the moment of a figure about its end farthest from the origin.

By beginning at the y axis and integrating the area under the moment curve, the triple integral of the function that originally represented the shape of the figure is obtained. The final ordinate to this curve, Fig. 1(C), may be called Y, and

$$Y = \int\int\int f(x)\,dx^3 \quad . \; . \; . \; . \; . \; . \; . \quad [3]$$

The moment of inertia (I_A) of the section in Fig. 1 about its end farthest from the x axis may be expressed as

$$I_A = \int (a - x) f^2(x)\,dx \quad . \; . \; . \; . \; . \; . \quad [4]$$

Integrating the expression by parts,

$$\int u\,dx = u\,x - \int v\,du \quad . \; . \; . \; . \; . \; . \quad [5]$$

Let $u = (a - x)^2$; $\quad du = -2(a - x)\,dx$;

$dv = f(x)\,dx$; and $\quad v = \int f(x)\,dx$

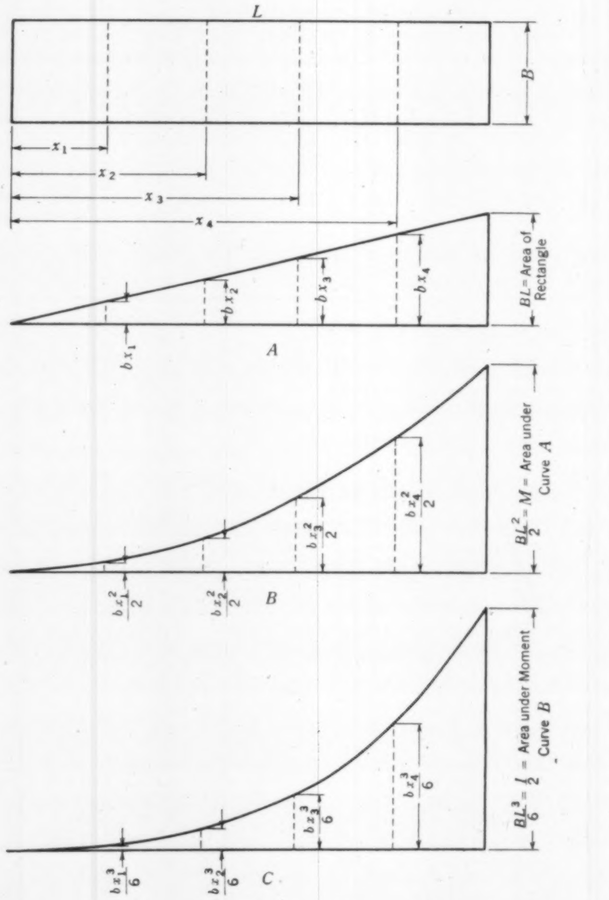

L
B
x_1
x_2
x_3
x_4
bx_1
bx_2
bx_3
bx_4
BL = Area of Rectangle
A
$\frac{bx_1^2}{2}$
$\frac{bx_2^2}{2}$
$\frac{bx_3^2}{2}$
$\frac{bx_4^2}{2}$
$\frac{BL^2}{2} = M$ = Area under Curve A
B
$\frac{bx_1^3}{6}$
$\frac{bx_2^3}{6}$
$\frac{bx_3^3}{6}$
$\frac{bx_4^3}{6}$
$\frac{BL^3}{6} = \frac{I}{2}$ = Area under Moment Curve B
C

Fig. 2. Application of Graphical Method to a Rectangle

Placing these values in Equation 4, there results

$$\begin{aligned} I_A &= \int (a - x)^2 f(x)\,dx \\ &= (a-x)^2 \int f(x)\,dx + 2\int [(a-x) \int f(x)\,dx]\,dx \\ &= (a - x)^2 \int f(x)\,dx + 2[\int (a - x) \int f(x)\,dx^2] \end{aligned}$$

Now let $u = (a - x)$; $\quad du = -dx$;

$$dv = \int f(x)\,dx; \quad \text{and} \quad v = \int\int f(x)\,dx^2$$

From these values the following equation is obtained:

$$I_A = \int (a - x)^2 f(x)\,dx = (a - x)^2 \int f(x)\,dx + 2\int\int\int (a - x)\,f(x)\,dx^2 + 2\int\int\int f(x)\,dx^3$$

In any case in which this method is used, the limit of x equals a. Then, placing the limits in the equation just given, the value of the moment of inertia is obtained:

$$I_A = 2\int\int\int f(x)\,dx^3 \quad . \;.\; .\; .\; .\; . \quad [6]$$

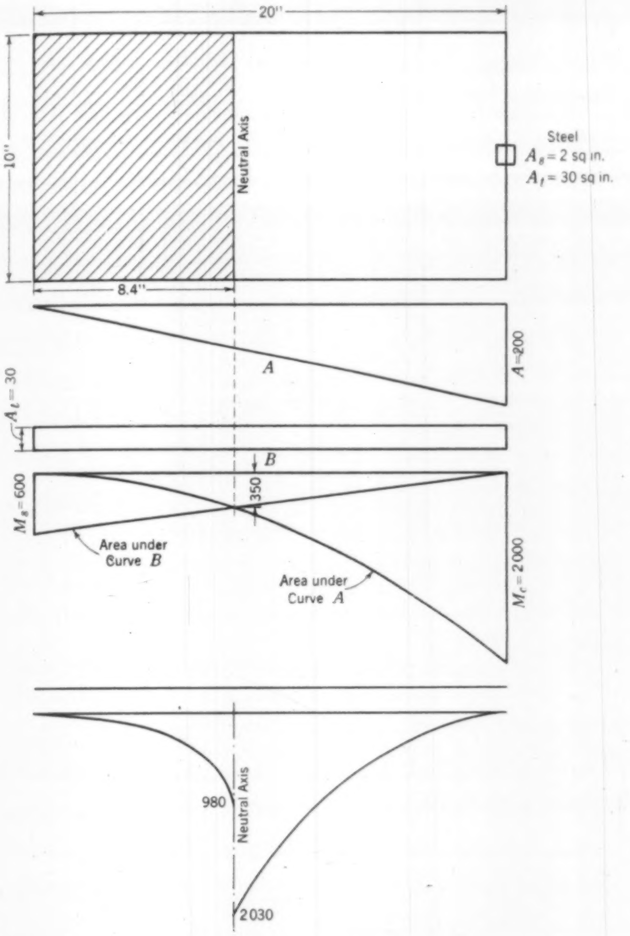


FIG. 3. GRAPHICAL METHOD APPLIED TO A BEAM OF TWO MATERIALS

Equation 6 may now be compared to Equation 3. In this comparison I_A equals $2Y$. The important point to remember is that the area of the moment diagram, D, in Fig. 1, will equal one-half the moment of inertia of the figure about its end and will be true for all figures.

Two simple cases will suffice to indicate the many uses to which the method may be put. In the first case, assume that it is desired to find the moment of inertia of a rectangle about its base, or $I = \frac{bL^3}{3}$. The steps are indicated in Fig. 2.

The moment of resistance of beams of two materials can be found, as indicated in Fig. 3. Assume that it is desired to find the moment of resistance of a rectangular concrete beam 10 in. wide and 20 in. deep to the steel, in which n equals 15; f_c equals 800 lb per sq in.; and f_s equals 16,000 lb per sq in. The curves representing the integrals of the area and moment of the rectangular concrete section are plotted the same as in the first case. The transposed area of the steel is 30 sq in. Since the steel is concentrated at a point, the integral of the area will be a horizontal line 30 sq in. high, as shown in B of Fig. 3.

Superimpose the integral curve of the area under the "area curve" producing the moment curve on the concrete moment curve. The two curves cross at a point 8.4 in. from the top of the beam, where both moment curves have a value of 350 cu in. The crossing of the moment curves thus marks the neutral axis of the beam. The area under the concrete moment curve up to the neutral axis equals 980 in.^{4}, and the area under the steel moment curve up to the neutral axis equals 2,030 in.^{4} These values equal one-half the moment of inertia of the concrete and steel about the neutral axis. The moment of inertia of the transposed section then becomes

$$I = 2\,(980 + 2{,}030) = 6{,}020 \text{ in.}^4$$

Using the flexure formula, the moment of resistance of the beam can be calculated as follows:

$$\text{Concrete } M_R = f_c\,\frac{I}{C} = \frac{800 \times 6{,}020}{8.4} = 573{,}000 \text{ in.-lb}$$

$$\text{Steel } M_R = \frac{f_s\,I}{n\;c_2} = \frac{16{,}000}{15}\;\frac{6{,}020}{11.6} = 554{,}000 \text{ in.-lb.}$$

The latter governs.

In addition to its usefulness in finding the moment of inertia of odd-shaped members made of one or more materials, I have found the method helpful in solving problems by the column analogy, as proposed by Hardy Cross, M. Am. Soc. C.E. The areas of the member, area diagram, and moment diagram may be obtained by the slope method, by the pole and ray method, by tabular form, or by direct integration. One example of such use was given in an article by Brent C. Jacobs, M. Am. Soc. C.E., on page 216 of the April issue of CIVIL ENGINEERING. For making such calculations I have found useful an inexpensive integraph manufactured by the Barr Integraph Company, of Cleveland, Ohio. The Jacob integraph is another such instrument. More expensive integrating instruments of foreign manufacture are also available.

Concrete Quantities by Absolute Volume Method

By W. J. HARRIS, ASSOC. M. AM. SOC. C.E.

ENGINEER OF MATERIALS AND TESTING, LOS ANGELES COUNTY ROAD DEPARTMENT, LOS ANGELES, CALIF.

FROM the weight of materials and their specific gravities, the absolute volume method of calculating quantities for portland cement concrete mixes determines the exact amount of space these materials will occupy in the completed concrete. The usual method is to express the mix in proportions by volume and to determine the absolute volume of each from the unit weight of the various ingredients. The summation of these volumes gives the quantity of concrete produced by one sack of cement.

In modern concrete construction, it is frequently the practice to specify the combined grading of the aggregate and the amount of cement per cubic yard for the various classes of concrete. Aggregate is usually furnished in several sizes: large rock, medium-sized rock, pea gravel, and sand. From the screen analyses of these various materials is determined the percentage of each needed to produce the combined grading specified.

It then becomes necessary to compute the weight of aggregate required to produce a batch of concrete of the desired size. To simplify this calculation I have evolved the following solution.

Let W = weight of the aggregate in pounds per cu yd of concrete

P = percentage of the aggregate consisting of sand

R = specific gravity of rock

S = specific gravity of sand

C = sacks of cement per cubic yard of concrete

G = gallons of water per sack of cement

Assuming that water, at 7.48 gal per cu ft, weighs 8.33 lb per gal, then one cubic foot of water weighs 62.31 lb. Assume also the weight of one sack of cement to be 94 lb and its specific gravity to be 3.1, then the absolute volumes of the various materials in a cubic yard of concrete are as follows:

$$\text{Cement} = \frac{94\,C}{3.1 \times 62.31}\ \text{cu ft}$$

$$\text{Water} = \frac{C\,G}{7.48}\ \text{cu ft}$$

$$\text{Sand} = \frac{W\,P}{62.31\,S}\ \text{cu ft}$$

$$\text{Rock} = \frac{W\,(1-P)}{62.31\,R}\ \text{cu ft}$$

Adding these expressions together and equating to 27 cu ft,

$$\frac{W\,(1 - P)}{62.31\,R} + \frac{W\,P}{62.31\,S} + \frac{C\,G}{7.48} + \frac{94\,C}{3.1 \times 62.31} = 27$$

or

$$W = \frac{R\,S}{R\,P + S\,(1 - P)}\,[1{,}682.4 - C\,(30.3 + 8.33\,G)] \quad \ldots\ldots [1]$$

Two unique relationships are to be noted in this equation. First, the expression within the brackets is the weight of a volume of water equivalent to the absolute volume of the sand and rock. Second, when R equals S, that is, when rock and sand have the same specific gravity, the expression $\frac{R\,S}{RP + S\,(1 - P)}$ reduces to R or S.

In many localities, the average specific gravity of both rock and sand is 2.65. Substituting in Equation 1,

$$W = 4{,}458 - C\,(80.3 = 22.1\,G) \ldots\ldots [2]$$

For example, given 6 sacks of cement per cu yd and 5 gal of water per sack of cement, $W = 4{,}458 - 6\,[80.3 + (22.1 \times 5)] = 3{,}313$ lb.

It is interesting to calculate, by means of a simple substitution in Equation 2, the effect of additional water on the quantity of aggregate. Substituting 7 gal for the 5 gal used in the foregoing example, we have $W = 3{,}048$ lb, a loss in aggregate of 265 lb, which is equivalent in absolute volume to the 12 gal of water added.

These formulas provide a simple and accurate method of calculating quantities of concrete materials and by eliminating the necessity of expressing the mix in proportions of volume and determining the unit weights of each, greatly reduce the labor of making the calculations and the possibility of errors in them.

Our Readers Say—

In Comment on Papers, Society Affairs, and Related Professional Interests

Distribution of Shear in Continuous Slabs

DEAR SIR: Professor Morris's excellent report on "Concentrated Loads on Slabs," a résumé of which appears in the March issue, is confined to the analytical and experimental determination of moments and shears in a rectangular slab simply and rigidly supported on two or three sides. Although this information is of undoubted value to the engineer engaged in designing bridge floor slabs, it is evident that an analytical and experimental investigation of the effect of concentrated loads on slabs typical of bridge construction, where the slab is continuous over the flexible steel stringers for a distance of four or more 5-ft spans and rests on stiff floor beams spaced twenty or more feet apart, would serve to settle many as yet unanswered questions involved in the design of such slabs.

Partial analytical solutions of the problem have been indicated by some writers, and at least one notable series of tests on a continuous slab was performed in connection with the design of the Delaware River Bridge floor. These tests were made on a specially reinforced 6-in. concrete slab, continuous over six steel stringers, but stopped short of the floor beams. The application of a central concentrated load to this slab produced smooth deflection curves, which showed that the stringers deflected with the slab and that the negative moments over the stringers were greatly reduced from those for rigid supports. The maximum longitudinal compressive stresses in this slab were found to be approximately twice as great as the maximum transverse compressive stresses, possibly due, also, to the deflection of the stringers. Important constants that should be determined in an investigation of the continuous bridge slab are the effective width of slab and the correct coefficients of longitudinal and transverse movement for various loading conditions and positions.

With the thinner slabs now coming into use on bridges, shear and diagonal tension become important factors. The report discusses the distribution of shear to the stringers and to the end support due to central loads, but not due to loads near the stringers. It is also stated that all the slabs failed by punching shear. Tests to determine the distribution of maximum shear in the continuous slab near the stringers and floor beams would be very useful. It seems probable that the effective width of slab in resisting shear would be much smaller for loads near a support than in the center of the span.

C. H. GRONQUIST, Assoc. M. Am. Soc. C.E.
Robinson and Steinman

New York, N.Y.
July 6, 1934

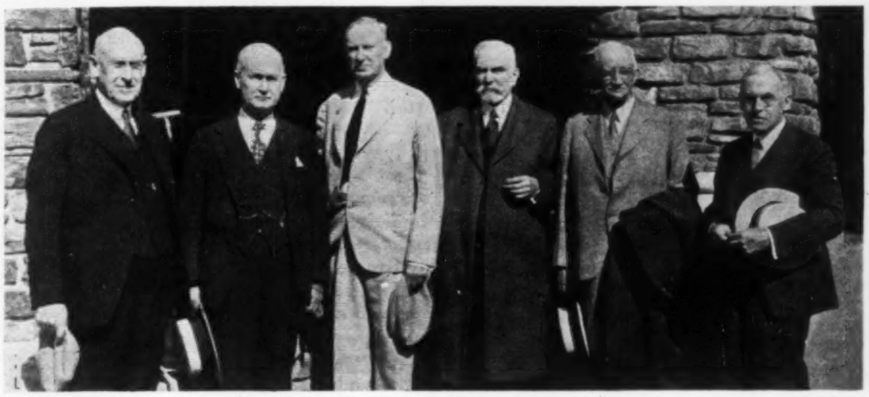
SOCIETY AFFAIRS

Official and Semi-Official

Vancouver Entertains

Annual Convention in Canadian City a Pronounced Success

Promotion of international cordiality and the exchange of engineering experiences on important projects of two countries were the dominating features of the Annual Convention held in Vancouver, B.C., July 11–14, 1934. Although it was expected that the Canadian city would "do itself proud," the results as evidenced by the enthusiasm of the many engineers in attendance, both Canadian and American, were even beyond the preliminary expectations.



Present and Former Officers of the Society at Banff, Alberta

Left to Right: Harrison P. Eddy, President; Alonzo J. Hammond, Past-President; F. O. Dufour, Vice-President; Otis E. Hovey, Treasurer; Arthur N. Talbot, Past-President; and George T. Seabury, Secretary

In part this was due to the excellence of the program, but it was also very largely the result of the fine spirit of cooperation and friendly interest displayed by both the Engineering Institute of Canada and the Society. The results were a credit to both organizations and left a deep impression of mutual appreciation on all sides.

At the very outset of the Convention, at the Wednesday morning meeting, the exchange of felicitations set the standard for the happy days to follow. Official greetings of cordiality were followed by the President's annual address, delivered by Harrison P. Eddy and a delightful descriptive lecture on the scenic possibilities of a trip to Alaska. The afternoon of that day was devoted to a thorough discussion of the Columbia River Drainage Basin Project by both Canadian and American engineers. This embraced a wide territory, including the northern reaches of the Columbia River in Canada, the Grand Coulee development, and the Bonneville Project.

Technical Divisions Meet

Thursday was given over to a variety of stimulating technical discussions covering many fields. On the part of the Society, the Technical Divisions held the interest with joint sessions sponsored by the Highway and Construction Divisions and by the Irrigation and Power Divisions both morning and afternoon, and a separate session of the Sanitary Engineering Division in the afternoon only. Although the subjects presented were of a scientific rather than a popular nature, keen interest was shown in all of them, as evidenced by the attendance and discussion. On the same day, during both the morning and afternoon, the Western Professional Meeting of the Engineering Institute of Canada was being held in adjoining rooms.

All the gatherings of the day were open without reserve, so that there was great difficulty in deciding what sessions to attend.

Varied Entertainment Provided

Through all the technical activities, a program of social events was interspersed. This included a luncheon on Wednesday and a dinner and entertainment that evening, both held at the Vancouver Hotel. Meanwhile, also, the ladies were being royally entertained by drives, teas, and sight-seeing trips. One of the high lights of the social program occurred on Thursday evening when the entire party enjoyed an informal dinner dance at the Grouse Mountain Chalet, high on a mountain overlooking Vancouver. Coming at the close of the only rainy day of the week, this trip seemed rather doubtful of success. But during the evening the clouds surrounding the mountain disappeared, and the myriad lights of the city below gave an effect of almost fairy-like beauty. The glorious setting, the splendid entertainment, and the delightful trip to and from the chalet made this evening unforgettable.

New pleasures were in store the following day when both the Canadian and American engineers enjoyed a long trip to Powell River, north of Vancouver, a distance of about seventy miles. Embarking at nine o'clock, the party discovered fresh wonders of Canadian beauty throughout the day. At Powell River the large paper mills and power plant were inspected with much interest. The organization of and accommodations for the trip were all that could be desired, and the return in the late evening was most picturesque. For a combination of delightful meals, excellent company, and panoramas of natural beauty, this event would be hard to duplicate.

Plans for the last day, Saturday, July 14, included a trip to the power plant at Ruskin, B.C., a game of golf for those who preferred that diversion, and an excursion to Seymour Canyon and Falls in the afternoon.

Many were the expressions of regret as the time of parting drew near. The thoughtfulness and efficiency of the local members were everywhere in evidence. The Vancouver Hotel and the Canadian Pacific Railway also cooperated in making the meeting and trips a great success.

Among the attendant events should be mentioned the special tour in advance of the Convention enjoyed by Board and other Society members and their families to the number of forty or more. Particularly the two days of the trip spent in the Canadian Rockies around Banff, Lake Louise, and the Yoho Valley were memorable. The party continued almost intact to Vancouver, arriving on Monday morning so that the Board of Direction might devote two days to Society business preceding the Convention.

Record of Attendance

A record of the approximate attendance at the various technical sessions and social events follows:

Total registration, combined societies	475
Wednesday morning:	
General Meeting	225
Wednesday afternoon:	
General Technical Meeting	200
Ladies' drive	115
Wednesday evening:	
Dinner and entertainment	210

Thursday morning:	
Session of Irrigation and Power Divisions	100
Session of Highway and Construction Divisions	50
Thursday afternoon:	
Session of Irrigation and Power Divisions	60
Session of Highway and Construction Divisions	45
Session of Sanitary Engineering Division	30
Thursday morning and afternoon:	
Technical sessions of the Western Professional Meeting of the Engineering Institute of Canada	75
Thursday evening:	
Informal dinner dance	230
Friday:	
All-day boat trip to paper and pulp mill	215
Saturday:	
Trip to British Columbia Electric Power Plant	100
Excursion to Seymour Canyon and Falls	125

Members of the Society have come to expect great things of meetings held within the Dominion of Canada, and the Vancouver Convention fully maintained this tradition. In fact it augmented the high esteem in which the sister country and its engineering organizations are held and in every respect set a high standard for similar gatherings of the future.

It is expected that abstracts of the various papers will be published in the September issue of CIVIL ENGINEERING. Members who were not fortunate enough to be able to attend the Convention and hear the papers presented in person will thus have an opportunity to read them.

Meeting of the Board of Direction—Secretary's Abstract

ON JULY 9 and 10, 1934, the Board of Direction met at the Hotel Vancouver, Vancouver, B.C., Canada, with Harrison P. Eddy, President, in the chair; and present George T. Seabury, Secretary; Otis E. Hovey, Treasurer; and Messrs. Ammann, Barbour, Black, Dewell, Dufour, Enger, Etcheverry, Gregory, Hammond, Hoffmann, Hogan, Horner, Jonah, Lupfer, McDonald, Noyes, Perry, Reed, Riggs, Sherman, Stevens, Trout, and Wilkerson.

Approval of Minutes

The Board approved the minutes of its meetings held on January 15, 16, on January 18, and on May 11, 1934.

The Board approved the minutes of the meeting of the Executive Committee held in Washington, D.C., on March 23, 1934, and adopted its actions as the actions of the Board.

Death of Past-President Grunsky

The Board observed a moment of silence in respect to the memory of Past-President Grunsky, who died on June 9, 1934, and authorized the President to appoint a committee to prepare his memoir.

Society Unit to Assist Members Unjustly Accused

The Board authorized the appointment of a committee of three Board members to report to it at its fall meeting the practicability of devising a method of procedure whereby members who may have been unjustly accused or condemned without proper hearing or deprived of position or recompense unjustly may have recourse to some Society unit which would endeavor to be of assistance to them.

Reports Received

Reports were received from representatives on the United Engineering Trustees, the Engineering Foundation, and the Society's research, professional, and administrative committees.

Report from Coordination Committee of Engineering Societies

The Board was in receipt of a report from the Coordination Committee of Engineering Societies suggesting a National Advisory Board of Engineers. The subject was tabled for further consideration.

New Societies to Be Studied

The President was authorized to appoint a committee of three members of the Board of Direction to report to it at its fall meeting with respect to the several movements in different parts of the country looking toward the formation of new societies of engineers, each primarily based upon an effort to protect the positions held by engineers, their salaries, and other conditions incident to their method of employment. The President appointed Directors J. P. H. Perry, New York; Frederick H. McDonald, Atlanta; and Frank A. Barbour, Boston, to that committee.

Relations of the Federal Relief Agencies to Engineers

At the Board meeting in January a committee composed of the President of the Society, the Secretary of the Society, and Edwin F. Wendt, M. Am. Soc. C.E., of Washington, D.C., was appointed to establish relations, if practicable, with the Federal authorities, with a view to seeing what could be done to rectify the conditions with respect to the very low salaries which had been paid to engineers under the CWA.

President Eddy reported that he had been fortunate in making very happy contacts with the authorities of the Federal Emergency Relief Administration; that they had accepted the Report on Prevailing Salaries of Civil Engineers adopted by the Board on March 6, 1934, as a criterion by which salaries should be paid to civil engineers and had endorsed it to their regional and local representatives. He reported also that where local conditions had been reported to be unsatisfactory, the committee had taken them up with the Federal authorities at Washington, which had given support to a prompt improvement.

New Student Chapter

The Board approved the reinstatement of the Student Chapter at the University of Mississippi, which had necessarily been discontinued at the time, some two or three years ago, when that university had temporarily lost its standing as an accredited school by action of the Board. The University of Mississippi was reinstated as an accredited school at the January Board meeting.

Local Section Constitutions

The Board approved amendments to the constitutions of the Mid-South, Duluth, Central Ohio, Arizona, and Chattanooga Sections, the amendment to the latter constitution being one by which the Chattanooga Section will be known as the Tennessee Valley Section, with branches at Knoxville and Chattanooga, Tenn., Florence, Ala., and Asheville, N.C.

Future Meetings

The Board decided it would hold its next meeting at Chicago in the latter part of September or early in October.

The Board decided to hold no Spring Meeting of the Society in the year 1935 and to hold the 1935 Convention at some point not yet determined but not east of Colorado.

Members in Arrears

The Board adopted a procedure similar to that adopted in previous years with respect to the dues of those in arrears.

Minimum Definition of Engineer

The Board approved the minimum definition of engineer as recommended to it and the other cooperating bodies by the Engineers' Council for Professional Development. The definition approved appears elsewhere in this issue.

Possible New Grades of Membership

The Board authorized a committee of three members of the Board to report to it at its fall meeting such steps as it might take looking toward acceptance of the suggestion by the Engineers' Council for Professional Development that there be more uniformity in the lower Corporate Member grade in the several societies and the possible establishment of a grade of "Student Member" and one of "Fellow." The personnel of this committee also will be Directors Perry, Barbour, and McDonald.

Mapping and Local Control Surveys

The Board adopted two resolutions endorsing the local-control survey work done last year under the Civil Works Administration. One resolution urged upon the Federal Emergency Relief Administration immediate planning for the adoption of a similar program for the coming fall, winter, and early months of next year. The other urged upon the Public Works Administration the making available of as much money as could be used practicably for that purpose and the prompt development of more base maps. These resolutions appear elsewhere in this issue.

Politics and Engineering Appointments

The Board adopted a resolution deploring the entrance of political party considerations into appointments in engineering positions. The Board particularly deplored the example of which it had knowledge set within the Bureau of Reclamation.

Hotel Water Supply to Be Certified to Prior to a Society Meeting

The Board voted that in future, before determining upon the place in which it would hold its meetings or those of the Society, it would obtain a satisfactory assurance of the condition of the water supply in accordance with similar procedure initiated by the American Water Works Association as a result of the unfortunate circumstances incident to local epidemics of amoebic dysentery.

Government Competition with Private Practice

The Board directed the Secretary to inquire as to the extent, or the policies governing the extent to which Federal bureaus have in mind the negotiation of contracts with political subdivisions for the engineering services which the latter may need.

Report on Salaries Revised

The Board had before it a communication from the chairman of the Society's Committee on Salaries wherewith he transmitted a porposed substitute for the report adopted by the Board on March 6, 1934, entitled "Prevailing Salaries of Civil Engineers." The Board reviewed a digest of the comments which had been received in regard to the report and voted to substitute the newly received report for the earlier one, and further instructed the officers of the Society to secure general recognition of this new report. It appears in full elsewhere in this issue.

Code of Fair Practice

The Code of Fair Practice for the Engineering Division of the Construction Industry, under consideration by NRA officials, was discussed in the light of an extensive memorandum. The action of the Executive Committee in submitting the code and asking for its approval was approved by the Board. Copies of the memorandum can be obtained if desired.

Report of Committee on Professional Conduct

The Board received and acted upon a report from its Committee on Professional Conduct dealing with five matters.

Question of Addition to Code of Ethics

The Board rescinded the action of the 1933 Board with respect to a new Paragraph 7 proposed to be added to the Society's Code of Ethics in order that the matter might be referred back to the Committee on Professional Conduct for further consideration.

Federal Housing Administration

By instruction of the Board the Secretary is to offer the services of the Society in any way that may be found to be helpful and practicable to the new Federal Housing Administration.

Automobile Taxes to Be Restricted to Highway Uses

The Board, by resolution, urged the membership of the Society to support in the several states legislative and administrative measures looking toward the restriction of gasoline and other automobile taxes to the cost of engineering, construction, maintenance, and patrol of the expanding system of highways only.

J. Waldo Smith Legacy

The $20,000 legacy bequeathed to the Society by the late J. Waldo Smith, Hon. M. Am. Soc. C.E., having been received, the Board of Direction, by formal resolution, pledged the Society to carry out the terms of the bequest in accordance with the conditions and for the purposes stated.

Election of Secretary, Treasurer, and Assistant Treasurer

The present incumbents were reelected: respectively, George T. Seabury, Secretary; Otis E. Hovey, Treasurer; and Ralph R. Rumery, Assistant Treasurer.

Budget

The Society's financial situation was reviewed, it being found practicable to make further appropriations to several of the Society's technical and professional committees.

Adjournment

Routine administrative matters were considered, and the Board adjourned at 10:45 p.m., Tuesday, July 10, 1934, to meet in Chicago at a time to be fixed by the Executive Committee, preferably late in September or early in October.

Diversion of Gas Tax Decried in Board of Direction Resolution

FOLLOWING is the text of the resolution adopted by the Board of Direction at its meeting in Vancouver, voicing its criticism of the unjustifiable use of gasoline taxes for purposes other than highway construction, maintenance, and patrol:

WHEREAS, A Western state initiated the policy of levying a special tax on automobile users in order to produce a continuing income to meet the cost of engineering construction, maintenance, and patrol of the expanding system of highways, and

WHEREAS, Because of the equitable apportionment of this tax and of the general satisfaction with the resulting expenditure, the plan and policy has been almost universally adopted, and

WHEREAS, The payment of such a tax has received the approval of automobile owners so long as the funds have been made available for highway development only, and

WHEREAS, Because of the large yield of this tax and the simplicity of its collection there has been in recent years and in several states an unreasonable increase in the rate of said taxes and a diversion of the revenues to other public institutions or to operations unrelated to the highway, and

WHEREAS, The recent congressional enactment making Federal funds available for highway development, by reducing the possible allotment to those states that have not restricted the use of such revenues to highway purposes, is the first statement of a national policy disapproving such diversion, now therefore be it

Resolved, That the Board of Direction of the American Society of Civil Engineers, assembled in regular session, July 10, 1934, approves said policy, contained in the Cartwright Bill, and urges the membership of the Society to support legislative and administrative measures of a like purpose in the several states.

Honoring Memory of Carl E. Grunsky

BECAUSE of his deep interest in Society affairs, his wide acquaintance among engineers, and his efforts for all worth-while projects, whether civic, cultural, or technical, Carl E. Grunsky was beloved far and wide, and especially in his own state of California. The affection with which he was regarded in Sacramento, where he lived for many years, is apparent from the resolution that follows, adopted by the Sacramento Section shortly after his death on June 9, 1934.

"The career of Carl Ewald Grunsky, Member and Past-President of the American Society of Civil Engineers, was such that the members of his profession have been delighted to do him honor as an eminent engineer, a distinguished scholar, a worthy citizen, and above all a gentle and gallant gentleman. In his passing the members of Sacramento Section experience a feeling of personal loss.

"Be it resolved therefore by the members of Sacramento Section, American Society of Civil Engineers, meeting in regular assembly this twelfth day of June, Nineteen Hundred Thirty-Four, that they express to the members of his family their appreciation of the service which he has rendered and regret that Time could no longer stay its hand."

Revised Report on Salaries of Civil Engineers

Improvements Incorporated in New Form Approved by Board of Direction July 10, 1934

DURING the Government's early efforts for relief by means of public works, criticism was made of the unreasonably low rates of pay for civil engineers. The objections of the Society met the logical rejoinder that there seemed to be no authoritative basis for determining the proper rates. This condition appeared to the Society officers as a challenge to action. Within ten days data previously accumulated by the Society's Committee on Salaries were codified in the form of a report, which appeared in the April 1934 issue of CIVIL ENGINEERING.

Whatever may have been the shortcomings of that document—and there were several which appeared on further study—none the less it can be said that it supplied a definite need at a critical time. It was promptly accepted as authoritative, and improvement in engineers' salaries was immediately effectuated by Government authorities. It had a second beneficial result in that it forthwith focused the attention of Society members. As a result many constructive suggestions were submitted, mainly from Local Sections. These suggestions led to the revision of the report in the form here given.

In studying the report it is important to note the limitation that the indicated 1934 salaries have particular relation to present economic conditions. For other times and circumstances material variations are to be expected, as evidenced by the salaries given in the report as having been paid in 1930.

The revised report is printed here in its entirety. It supersedes the previous form and is believed to be a material improvement. As such it was approved by the Board of Direction at its meeting in Vancouver and ordered to be thus distributed.

THIS report is an effort to determine from statistical data the salaries of Civil Engineers. It is based on the surveys of salaries made by the Committee on Salaries of the American Society of Civil Engineers in 1930 and again in 1933 on reports recently received from Local Sections of the Society and from many other sources. It is intended to set forth the prevailing rates of salaries of Civil Engineers as of the first quarter of 1934.

METHOD

The salaries paid to 16,046 engineers and engineering assistants in the highway departments of the 48 states in 1930 are taken as the basis of the calculations. Median salaries have been determined from numerous plottings. Fifty per cent of the engineers received these salaries or more. Fifty per cent received less. Seven classifications of positions are adopted, and the median salary for each has been determined. Combining these, a schedule of salaries for the different classifications is set up as an index. This index has been carefully compared with other indices for previous years and with actual salaries of 28,400 engineers under other conditions of employment. The effect of various geographical and social conditions permits the derivation of factors whereby the index schedule may be used in the calculation of salary rates.

REGIONS

Local cost-of-living conditions and other factors permit the division of the United States into regions over which these, or other, conditions indicate a variation in the salaries paid. The Bureau of Public Roads of the U. S. Department of Agriculture recognizes nine such regions and these same regions have been adopted in this study. These regions are:

New England: Maine, New Hampshire, Vermont, Massachusetts, Rhode Island, Connecticut.

Middle Atlantic: New York, New Jersey, Pennsylvania.

East North Central: Ohio, Indiana, Illinois, Michigan, Wisconsin.

West North Central: Minnesota, Iowa, Missouri, North Dakota, South Dakota, Nebraska, Kansas.

South Atlantic: Delaware, Maryland, Virginia, West Virginia, North Carolina, South Carolina, Georgia, Florida.

East South Central: Kentucky, Tennessee, Alabama, Mississippi.

West South Central: Arkansas, Louisiana, Oklahoma, Texas.

Mountain: Montana, Idaho, Wyoming, Colorado, New Mexico, Arizona, Utah, Nevada.

Pacific: Washington, Oregon, California.

CLASSIFICATION OF POSITIONS

Information relating to the duties, responsibilities and salaries of the engineers of the highway departments permits a relatively simple classification of positions which further have been taken as the basis for salaries of Civil Engineers engaged in other than highway work. These classifications, for which key salaries have been determined, are defined as follows:

Classification 1

Duties: Under immediate supervision, to assist on engineering work in field or office; to make simple measurements and sketches; to make and check simple computations; to make tracings and simple drawings from sketches and record plans; to do neat lettering of various forms; to assist in the compilation of engineering data; to assemble and file engineering plans, specifications, and records; to perform the usual work of a Rodman, Chainman, Tracer, or Junior Draftsman.

Requirements: Education equivalent to graduation from a high school, with at least two (2) years of experience in office or field assisting on engineering work.

Classification 2

Duties: Under supervision, to perform technical work in the office or field requiring a knowledge of engineering practices and methods; to make working drawings and designs and prepare estimates; to make computations of a technical character; to perform minor surveys and construction inspection. This classification includes the positions usually called Junior Engineer and Levelman, and the lower grades of Draftsman and Inspector.

Requirements: Education equivalent to graduation from a college or technical school of recognized standing, plus about two (2) years of experience on engineering work; thorough knowledge of the care, operation, and adjustment of engineering instruments; ability to design simple engineering structures.

Classification 3

Duties: Under supervision, to take charge of a field party, a drafting squad, or inspection of construction; to make and check quantity and cost estimates; to lay out work and draw plans and detail drawings; to check working drawings and fabricator's details; to keep records; to make reports. This classification includes the higher grades of Draftsman and the positions usually called Transitman, Engineering Inspector, Detailer, Checker, Squad Boss, and Chief of Party.

Requirements: Education equivalent to graduation from a college or technical school of recognized standing plus about three (3) years of experience in engineering work; ability to exercise engineering judgment and skill; ability to plan operations in office or field.

Classification 4

Duties: To perform important engineering work in office or field requiring the exercise of independent engineering judgment and skill; to make studies, estimates and reports; to prepare designs and specifications; to supervise the work of draftsmen, of field parties, or of a group of inspectors on construction work. This classification includes the positions usually designated as Designer, Designing Engineer, or Special Inspector.

Requirements: Education equivalent to graduation from a college or technical school of recognized standing plus about five (5) years of experience in engineering work of which at least one year shall have been in responsible charge; eligible for corporate membership in one of the national engineering societies, or for license as a Professional Engineer.

Classification 5

Duties: To be in responsible charge of a minor engineering organization or to have immediate administrative and technical charge of a division (comprising several squads or parties in office or field) in a large organization; to plan, direct, and supervise the design or construction of engineering projects; to supervise the preparation of contracts and specifications and to be entirely responsible for the engineering construction and business operations of his division, bureau, section, or department; to have immediate responsible supervision of the work of draftsmen, designers, and assistant engineers. This classification includes the positions usually designated as Engineer, Resident Engineer, and Section Engineer.

Requirements: A first-class technical education plus about eight (8) years of professional experience in the active practice of engineering; ability to plan, design and supervise engineering works.

Classification 6

Duties: To be in responsible professional and executive charge of a major division of work on a large engineering project, involving planning, designing, and supervision of construction; to give independent critical engineering advice for executive action incidental to such work; to assume professional and executive responsibility for the whole of a smaller engineering project. This classification includes the positions usually designated as Division Engineer, Senior Engineer, Assistant Chief Engineer, or Chief Engineer (of a minor organization).

Requirements: A first-class technical education supplemented by cultural and economic studies, plus about eleven (11) years of broad professional experience embracing a thorough knowledge of engineering office methods, field work, design, construction, and business administration.

Classification 7

Duties: To have entire charge of and be responsible for entire organizations comprising more than one engineering division and engaged in more than one class of work; to determine and establish technical and administrative policies and procedures; to be responsible for all engineering design, construction, and business operations; to investigate all engineering projects submitted for consideration and to report critically regarding feasibility, cost, economic justification, and public necessity or convenience. This classification includes the position of Chief Engineer of a major organization.

Requirements: A first-class technical education supplemented by cultural, economic, and social studies, plus about fifteen (15) years of broad professional experience embracing a thorough knowledge of engineering office methods, field work, planning, design, construction, maintenance, operation, engineering economics, contract law, and business administration; recognized standing in the engineering profession.

INDEX SCHEDULE

The index schedule combines the classifications of positions with the median salary for each classification. Salaries derived from the 1930 survey are shown by the later surveys to have suffered, in common with incomes from other sources, reductions differing locally but in general variable with relation to their size, that is, the larger reductions being observed in the higher salaries. The following is set up as the index schedule showing both actual median 1930 salaries and the actual reduced median salaries as of the first quarter of 1934. These salaries thus constitute "key" salaries, from which by the use of factors, salaries for other situations may be derived.

CLASSIFICATION	1930	1934
1	$1,330	$1,200
2	1,824	1,620
3	2,280	2,000
4	3,000	2,600
5	4,100	3,500
6	5,400	4,600
7	7,000	6,100

Fifty per cent of the engineers in each position classification in the schedule receive the salary shown or more. Fifty per cent receive less. As an example of the distribution of individual salaries within one particular classification, it is observed that of the salaries of Assistant Engineers (corresponding to Classification 4) in state highway departments,

Approximately 6 per cent receive about 28 per cent more than the median.
Approximately 22 per cent receive about 15 per cent more than the median.
Approximately 48 per cent receive about the median salary.
Approximately 20 per cent receive about 11 per cent less than the median.
Approximately 2 per cent receive about 23 per cent less than the median.

Detailed analysis of other classifications shows somewhat similar patterns.

REGIONAL FACTORS

To the indicated key salaries there should be added or subtracted amounts indicated by regional factors determined from early 1934 actual data. Further, to these figures, when so adjusted, other factors are applicable, dependent upon other conditions.

The regional factors for 1934 were found to be as follows:

New England	plus 3 per cent
Middle Atlantic	plus 15 per cent
East North Central	plus 6 per cent
West North Central	minus 5 per cent
South Atlantic	minus 15 per cent
East South Central	minus 15 per cent
West South Central	minus 10 per cent
Mountain	minus 10 per cent
Pacific	plus 5 per cent

In local jurisdictions where it can be shown that the costs of living have not changed from 1930 to 1934 in accordance with this table, the proportionate factor may be employed subject to further modification by population or other factors.

POPULATION FACTOR

The surveys indicate that for those employed in cities of 1,000,000 population a further increase in salary is required to conform to prevailing rates in the amount of 10 per cent and that in New York City, with a population of approximately 7,000,000, a present factor of plus 30 per cent is applicable. Factors for other cities must be determined from local experience; the data available do not yet indicate these required factors for cities with other populations.

TYPES OF EMPLOYMENT

The above index schedule, adjusted by regional and population factors, applies to engineers engaged in work similar to highway work and resident under similar conditions. Data relative to 28,400 engineering positions of different character justify increased salaries for those employed in certain other types of work or organizations. In general those in city employ are found to conform to the adjusted index schedule. For those many selected individuals, such as those in quasi-public employment, in industry, or in private organizations, the adjusted key salaries are to be subject to further increases, often in quite substantial amounts. Furthermore, it should be kept in mind constantly that the figures indicated in this report were all determined on the basis that 50 per cent of all the engineers regularly employed receive these adjusted key salaries or more, while 50 per cent receive less.

Federal engineers have had their 15 per cent reduction restored to the extent of 5 per cent on February 1, and 5 per cent on July 1, 1934.

CAUTION

Statistically the outlined system is believed to be justified, using the median salary as the primary basis. Such a system, however, cannot take into account unusual requirements or conditions, or matters of particular circumstance in respect to those engineers upon whom great responsibility is placed. There is no certain method of standard appraisal of the worth of intelligence, knowledge, experience, administrative ability, enthusiasm, and activity; it has not been possible to determine the amount of additional compensation now paid for them. The above analysis, however, is believed to afford a statistical basis to which an appraised value of these intangibles may be applied.

Base Mapping as Medium for Relief Activities

RESOLUTION OF BOARD OF DIRECTION INDICATES NEEDS AND OBJECTIVES

THE BENEFICIAL results to be expected from base mapping throughout the country coupled with the opportunity for unemployment relief prompted the Board of Direction to adopt a resolution, at its Vancouver meeting, as follows:

WHEREAS, There is urgent need for accurate base maps of the United States and political subdivisions thereof for the use of agencies engaged in national, regional, and local planning for governmental and private development, and

WHEREAS, Such maps are not available for large portions of the country, and

WHEREAS, The supplying of such maps is the normal and accepted duty of the Federal Government, and

WHEREAS, Unemployment of professional engineers is a grave and continuing problem of large proportions in many parts of the country, and

WHEREAS, Large numbers of engineers and assistants could be employed in the needed extension of the mapping activities of the Federal Government for local-control surveying and mapping, and

WHEREAS, Large funds expended for this type of work result in a maximum of individual employment, and

WHEREAS, The prosecution of such work would result in necessary and permanent benefits, therefore be it

Resolved, By the Board of Direction of the American Society of Civil Engineers assembled in regular session July 10, 1934, that the Federal Emergency Public Works Administration be urged to appropriate to a surveying and mapping program as large a sum as can be made available to be expended under the direction of the U. S. Coast and Geodetic Survey and the U. S. Geological Survey in a program of such fundamental mapping activities.

Approval of Minimum Definition of Engineer

AS EXPLAINED in previous issues, the Engineers' Council for Professional Development (E.C.P.D.) is a conference of engineering bodies organized to enhance the professional status of the engineer through cooperative support and coordinated programs looking toward better professional recognition. In line with this purpose, it has proposed to the Society and other organizations a minimum definition of an engineer—given on page 224 of the April 1934 issue—as one step in its avowed effort to secure this greater recognition.

Sympathetic to this purpose, the Board of Direction at its meeting in Vancouver approved such a minimum definition, which is here repeated:

"(*a*) Graduation from an approved course in engineering of four years or more in an approved school or college; a specific record of an additional four years or more of active practice in engineering work of a character satisfactory to the examining body (the examining body, in its discretion, may give credit for graduate study in counting years of active practice); and the successful passing of a written and oral examination covering technical, economic, and cultural subjects, and designed to establish the applicant's ability to be placed in responsible charge of engineering work and to render him a valuable member of Society; or alternatively

"(*b*) Eight years or more of active practice in engineering work of a character satisfactory to the examining body, and the passing of written and oral examinations designed to show knowledge and skill approximating that attained through graduation from an approved engineering course, and also examinations written and oral covering technical, economic, and cultural subjects designed to establish the applicant's ability to be placed in responsible charge of engineering work and to render him a valuable member of society."

Urging Continuance of Local-Control Survey Program

MEANS OF RELIEF OF PROFESSIONAL CLASS INDICATED IN BOARD RESOLUTION

WIDE BENEFITS derived from the Federal plan of local-control surveys in the several states, especially in relieving technical unemployment, have been patent to all students of affairs. Extension of this program during the year to come is the aim of the resolution adopted by the Society's Board of Direction in Vancouver, B.C., on July 10, 1934. This resolution states:

WHEREAS, The Federal Emergency Relief Administration in the fall of 1933 instituted a national program for the relief of professional unemployment by the provision of funds for state local-control surveys under the direction of the U. S. Coast and Geodetic Survey, and

WHEREAS, Said program was discontinued as a Federal project in February 1934, through restrictions placed by the Congress on funds then available for the employment of many engineers and assistants, which program, since that time carried on by local state action, has provided for the employment of a continuing but much smaller number, and has demonstrated the efficacy, soundness, and value of such a type of relief, and

WHEREAS, There does not appear to be prospect of a substantial reduction of professional unemployment by private enterprise during the approaching fall, winter, and spring months, and

WHEREAS, It does appear that large numbers of experienced and needy professional men will thus continue to be unemployed, and

WHEREAS, Funds are now available for relief and can be used for the execution of valuable and needed work as Federal projects, therefore be it

Resolved, That the Board of Direction of the American Society of Civil Engineers in regular session, July 10, 1934, in behalf of the

JOINT BANQUET OF THE ENGINEERING INSTITUTE OF CANADA AND THE SOCIETY AT THE VANCOUVER CONVENTION, JULY 11, 1934 engineering profession, expresses its appreciation of the efforts of the Federal Emergency Relief Administration to provide for professional employment and urges that the continuing problem of unemployment be anticipated by the prompt reinitiation of a nationwide program of state local-control surveys sufficient to provide for at least one year's employment of as large a number of engineers and assistants as it is practicable to employ under the direction of the U. S. Coast and Geodetic Survey in continuing the program initiated last fall.

A Preview of Proceedings

After the usual interval of two summer months during which Proceedings *does not appear, the next, or August issue will mark the resumption of publication. This number will contain three main papers, a symposium, and a wealth of discussion on 11 previously published papers. The symposium, on street thoroughfares, prepared by a committee of the City Planning Division of the Society, is an important contribution to the literature on city planning.*

Flow of Water Around Bends in Pipes

The paper on "Flow of Water Around Bends in Pipes," by David L. Yarnell and the late Floyd A. Nagler, Members Am. Soc. C.E., deals with experiments that are unique in that, with the same quantity of flow, the effect on the loss of head resulting from unequal velocity distribution in the pipe as it approached the bend, was fully investigated. In addition to standard pipe bends, numerous other special types were devised for special test purposes. The bend itself and 8 ft of the two tangents adjacent to the bend were made of transparent celluloid. Velocity traverses were made at different sections on the bend and on the two tangents.

On the basis of their findings, the authors conclude that: (1) all bends act as obstructions to flow, with consequent additional loss of head; (2) with uniform velocity distribution in the approach channel to the bend, the velocities of the filament along the inside wall are increased, while those of the filament along the outside wall are reduced; and (3) knowing the maximum difference in pressure between the inside and the outside of the bend, and the radius and size of the bend, it is possible to compute the mean velocity in the pipe, and hence the discharge.

Professor Nagler died on November 10, 1933, but the discussion will be answered in due course by Mr. Yarnell.

Eccentric Riveted Connections

In a paper at once brief and comprehensive, Eugene A. Dubin has developed general formulas for designing the rivets in eccentric joints in terms of one, two, three, and four rows of rivets. The formulas express, directly, the relationship between the loads, the eccentricity of the loads, the stress in extreme rivets, the spacing of the rivets, and the number of rivets in the group. As may be imagined, the resulting function of N, the number of rivets, is quite complicated and difficult to solve algebraically. To overcome this feature, the author indicates how the problem may be solved graphically by means of alignment charts. These charts, supporting four examples, constitute the most valuable part of the paper and should prove especially valuable to designers of structural steel work.

Determination of Trapezoidal Profiles for Retaining Walls

The Rankine theory of earth pressure is used widely by designers of retaining walls. Ordinarily, the process is a matter of "cut and try," the suitable section being guessed and then verified to see that it satisfies the conditions of the stress required. In his forthcoming paper, A. J. Sutton Pippard, M. Am. Soc. C.E., offers a method of determining the shape of the profile directly and quite rapidly. Examples are offered illustrating how the method is applied to walls with vertical and sloping backs, and with the earth fill level with the top, or surcharged.

Street Thoroughfares: A Symposium

On January 22, 1931, the City Planning Division created a Committee on Street Thoroughfares Manual, composed of W. W. Crosby, as chairman, and J. P. Callahan, Jacob L. Crane, Jr., Jay Downer, Wesley W. Horner, and Daniel L. Turner, all Members Am. Soc. C.E. George H. Herrold, M. Am. Soc. C.E., was sponsor of this committee and Contact Member with the Division. In the intervening years this committee has quite thoroughly analyzed and recorded its findings in a relatively new field. It submitted a progress report at the Annual Convention of the Society in Tacoma in July 1931. A second progress report was similarly presented at the Fall Meeting of the Society in St. Louis in October 1931. The symposium scheduled for the August issue of Proceedings is in the nature of a tentative final report. The purpose of its publication is to secure as widespread correspondence on this subject as is possible, so that all engineers who are interested in the general subject may have the opportunity of filing their reactions to the various papers and the report directly with the subcommittee. Subsequently it is the purpose of the committee to study this correspondence with a view to producing a final report for a Manual on this important subject. In accordance with the policy of the Society in regard to progress reports, all discussions will be transmitted directly to the subcommittee for its information.

In addition to the foreword by Mr. Herrold, and the report of the committee, the symposium includes six papers by individual members of the committee and a bibliography on street thoroughfares. In the order of their arrangement the titles of the parts of the symposium are as follows:

1. "Foreword," by George H. Herrold.
2. "General Recommendations: Progress Report of the Committee of the City Planning Division on Street Thoroughfares Manual."
3. "Relation of Street Thoroughfares to Other Arteries of Transportation," by Jacob L. Crane, Jr.
4. "Design of Boulevards and Parkways," by Jay Downer.
5. "Super-Highways," by J. P. Callahan.
6. "Thoroughfare Lay-Outs in Relation to Drainage Systems," by W. W. Horner.
7. "Express Highways and By-Path Routes," by Daniel L. Turner.
8. "Paying for Street Thoroughfares," by George H. Herrold.
9. "Bibliography on Street Thoroughfares," by the Committee of the City Planning Division on Street Thoroughfares Manual.

After the explanatory foreword by Mr. Herrold, the committee presents a series of well-considered and concise definitions with which all engineers should be familiar. This is followed by a general summary, with a statement of basic principles pertaining to the design of street thoroughfares. Mr. Crane's paper is confined to a studious discussion of the relation of street thoroughfares to other arteries of transportation, the more important arteries being as follows: (1) roadways or streets not classified as thoroughfares; (2) roadways and highways just outside the city; (3) footways; (4) railways, steam lines, and electric lines; (5) waterways; and (6) airways. The arguments presented lead inevitably to the conclusion that coordinated long-time planning is a vital necessity.

In this paper Mr. Downer stresses the fact that although parkways were originally created for esthetic reasons, they are now considered the most effective means of solving modern road problems on purely economic grounds. Dealing exclusively with super-highways, Mr. Callahan concludes that there is no economic reasons why the all-purpose roads of the future should not provide for esthetic treatment by landscaping as well as for utility needs, provided adequate right of way is secured from initial locations before the land acquires a prohibitive value as a result of the facilities furnished. The relation of a thoroughfare lay-out to the drainage system of a region is concisely defined by Mr. Horner, who states his firm conviction that proper original planning should provide a system of thoroughfares bordering natural drainage ways, to which would be joined a complementary system crossing valleys at the most advantageous points.

The subject of express highways and by-path roads as well as their accessories or supplements is treated by Mr. Turner, who prefaces his comments with an interesting prophecy by H. G. Wells written thirty years ago. Mr. Turner contributes the following definition of a super-highway: "A super-highway may be defined as a highway designed to accommodate mass passenger transportation facilities on rails and rubber tires, in addition to all other street uses."

In his paper on "Paying for Street Thoroughfares," Mr. Herrold gives the interesting and valuable historical background of the subject, describing the evolution of thoroughfares in the United States. The part of his paper that deals with the equitable distribution of construction costs may be truly classified as material of the utmost value to engineers engaged in municipal work.

The symposium ends with a bibliography containing fifty items which are considered the most important contribution to the literature of this specialized subject to date. No city planner, city or county engineer, surveyor, or other engineer whose sphere of activity includes official contact with municipalities and state governments, should fail to discuss this paper.

Appointments of Society Representatives

EDWARD J. MEHREN, M. Am. Soc. C.E., has accepted an appointment as a Society representative on the Commission of the Washington Award for the two-year term, June 1934–June 1936.

WALTER L. HUBER, M. Am. Soc. C.E., represented the Society at the meeting of the American Association for the Advancement of Science held in Berkeley, Calif., June 18–23, taking the place of the late C. E. GRUNSKY, Past-President Am. Soc. C.E., who had previously been appointed a Society delegate to this meeting.

News of Local Sections

BUFFALO SECTION

At a recent meeting of the Buffalo Section the following officers were elected for the ensuing year: Elwin G. Speyer, President, and Frederick O. Francis, Secretary.

DAYTON SECTION

The regular meeting of the Dayton Section for May took the form of an inspection trip to a broadcasting station at Mason, Ohio, and to broadcasting studios in Cincinnati. Members of the University of Dayton Student Chapter, as well as other guests, joined the Section for this trip. On June 18 the Section held a luncheon meeting at the Engineers Club. The feature of this occasion was a talk by Harold E. Winey, secretary-engineer of the Dayton City Plan Board, whose topic was "Dayton's Housing Projects."

LOS ANGELES SECTION

A dinner meeting of the Los Angeles Section was called to order on June 13 at the University Club, with 126 present at the dinner and a much larger number at the meeting that followed it. Among the speakers for the occasion were W. R. Gregg, chief of the U. S. Weather Bureau; Samuel B. Morris, superintendent and chief engineer of the Pasadena Water Department; and Prof. Graham Allan Laing, of the staff of the California Institute of Technology. These talks covered a wide range of engineering subjects and proved of great interest to the large audience.

MID-SOUTH SECTION

An interesting two-day session of the Mid-South Section was held in Memphis on May 25 and 26. Included in the business meeting was the annual election of officers, which resulted as follows: Clarence E. Boesch, President; W. E. Elam, Vice-President; and John H. Gardiner, Secretary-Treasurer. Included on the list of speakers were Lt.-Col. E. L. Daley, of the Corps of Engineers U. S. Army; George R. Clemens, senior engineer of the Mississippi River Commission; Kenneth Markwell, engineer for the Memphis Harbor Commission; and Capt. Hans Kramer, of the Corps of Engineers, U. S. Army. The two latter speakers explained the various features of interest in connection with Memphis Harbor, which was the objective of an inspection trip. The chief address given at the dinner held on the evening of May 25 at the Hotel Peabody was given by Charles N. Burch, Special Master for the U. S. Supreme Court at Memphis, who spoke on the subject, "Diversion of Delaware River Waters for New York City's Water Supply."

NORTHWESTERN SECTION

A meeting of the Northwestern Section, held at the University of Minnesota in Minneapolis on June 13, was addressed by M. F. Clements, for many years Construction and Bridge Engineer of the Northern Pacific Railroad. He gave an illustrated talk on the topic, "The Sand Island Method of Sinking Foundation Caissons," a construction method that was developed largely by him.

PHILADELPHIA SECTION

On June 20 the annual meeting of the Philadelphia Section was held at the Sandy Run Country Club near Philadelphia. About 30 played golf and engaged in other sports in the afternoon, and 45 attended a dinner meeting in the evening. The Section was honored by a visit from Harrison P. Eddy, President of the Society, who spoke most interestingly on Society activities and problems. Among the other speakers were Henry J. Sherman, Director of the Society, and Charles H. Stevens, former Director. During the business session the tellers announced that the annual election of officers for the coming year had resulted as follows: Isaac S. Walker, President; Lawrence E. Raymond and Berthold F. Hastings, Vice-Presidents; and Charles A. Howland, Secretary-Treasurer.

PITTSBURGH SECTION

At the annual meeting of the Pittsburgh Section, held on May 19, the following officers were elected for the coming year: John M. Rice, President; Charles L. Fox, Vice-President; and Nathan Schein, Secretary-Treasurer.

PORTLAND (ORE.) SECTION

A special meeting of the Portland (Ore.) Section was called on June 20 for the purpose of discussing the work of the National Planning Board of the Public Works Administration and for securing the cooperation of the Section in formulating local plans. Included among the speakers were Marshall N. Dana, chairman of District No. 11 of this board, and R. F. Bessey, technical advisor of the same district. The subject elicited wide discussion.

SYRACUSE SECTION

The following officers have been elected by the Syracuse Section for the coming year: E. F. Berry, President; V. J. Milkowski, First Vice-President; G. A. Helmstetter, Second Vice-President; and F. D. McKeon, Secretary-Treasurer.

Student Chapter News

CUSTIS LEE ENGINEERING SOCIETY (WASHINGTON AND LEE UNIVERSITY)

The Custis Lee Engineering Society Student Chapter met on March 26 to initiate eight new members into the Chapter. The program of entertainment consisted of an interesting talk by C. E. Sanford, student member, who spoke on the development of the diesel engine during the past year, and of the showing of a motion picture entitled "Electricity Goes to the Sea." This was furnished through the courtesy of the Westinghouse Electric and Manufacturing Company.

RHODE ISLAND STATE COLLEGE

There were 26 present at a meeting of the Rhode Island State College Student Chapter held on March 6. The manufacture of emulsified asphalt was described by C. L. McKesson, of the American Bitumuls Company. The lecture was followed by two reels of motion pictures, showing the process of constructing highways with this product.

ITEMS OF INTEREST

Engineering Events in Brief

The March of Progress

By Jasper O. Draffin, M. Am. Soc. C.E.

Associate Professor of Theoretical and Applied Mechanics, University of Illinois, Urbana

On the Page of Special Interest at the beginning of this issue are views of three unique bridges built in succession across the Connecticut River at Bellows Falls to connect the towns of Walpole, N.H., and Rockingham, Vt. Professor Draffin here gives the history of the site.

At the close of the Revolution, Col. Enoch Hale, prominent in the military and civil affairs of New Hampshire, secured a charter from the General Assembly and Council of his state granting him the exclusive right to build and maintain a toll bridge at "Great Falls," as Bellows Falls was then called. At that time the few residents of the district were engaged chiefly in farming, fishing, and in the carting of river freight from boats above and below the falls.

Colonel Hale's structure, completed in 1785, was the first bridge to be built over the Connecticut River and the first in this country to contain spans longer than could be bridged with simple wooden beams. He had selected an ideal site for his "foolhardy experiment," the narrowest point in the river, where there were high, steep banks and a rocky ledge in the middle of the stream for the pier. The floor of the bridge was at least 50 ft above the water, and the spans of four braced stringers are said to have been over 100 ft long. It is reported that the bridge cost $2,700. The view of Colonel Hale's bridge [on the Page of Special Interest] is reproduced from a painting said to have been done in 1793, although this date has been questioned. It probably shows the structure after rather extensive repairs had been made about 1800.

On the direct route between Boston and Albany and Boston and Montreal, the gorge and falls of the Connecticut River were as widely advertised to prospective stagecoach passengers in those days as was Niagara Falls to prospective travelers by rail in a later period. Referring to the location of the pier of Colonel Hale's bridge, a geography of 1805 stated: "On the steep sides of the island hang several armchairs, fastened to ladders and secured by a counterpoise, in which fishermen sit to catch salmon with dipping nets."

Tolls were deposited in a box made of birch bark sewed with linen thread and attached to a wooden bottom. The original box is said to be still in existence in a collection of Vermontiana. Bridge revenue mounted because of the high toll, ranging from 3 cents for foot passengers to 25 cents for mail stages, and many stages between Boston and northern New Hampshire avoided the toll by stopping at the New Hampshire end of the bridge, where the driver would blow his horn. Attendants from the inns on the opposite bank would meet patrons, pay the 3 cents toll for foot passengers, and transport trunks across on wheelbarrows.

Finally Colonel Hale found it necessary to raise some money by mortgaging the bridge to a wealthy Englishman, Frederic W. Geyer. When the mortgage became due, about 1800, his finances were still involved, but after considerable effort he raised the money and dispatched it by his son so that it would reach Boston the day before the time expired. Traveling by stage, the son stopped at a tavern over night where, it is said, he met his wife, from whom he had been estranged for some years. A reconciliation was effected, and in the excitement the important mission of saving the bridge was forgotten. The money arrived in Boston a day too late; Mr. Geyer refused to accept it and thus gained possession of the bridge.

During Mr. Geyer's ownership, the high tolls charged caused increasing dissatisfaction. The story is related that one particularly cold winter, when the ice was strong enough to carry the traffic, for several months not a vehicle crossed the bridge. Mr. Geyer attempted to prevent this loss by constructing a stone wall on the New Hampshire shore adjacent to the "Great Eddy," a short distance below the gorge, to prevent vehicles from getting down onto the ice. Citizens as promptly tore down the wall. Then Mr. Geyer retaliated by trying to ruin the road over the ice by placing salt on it, but by pouring water on the tracks immediately afterwards citizens kept the ice road open all winter.

At the death of Mr. Geyer, about 1820, possession of the bridge passed to his daughter, the wife of Nathaniel Tucker, and it became known as the Tucker Toll Bridge. In 1840, when Mr. Tucker found that the original structure was badly rotted, he replaced Colonel Hale's bridge by a two-span lattice-truss structure, built at the same location as the old bridge but 15 ft above it. The new bridge was 262 ft long over-all, provided for foot passengers, and was completely roofed over. When it was completed, on July 4 of that year, workmen cut away the old bridge underneath, which fell into the river after a useful life of 55 years. Nathaniel Tucker modified the tolls and sold annual tickets for five dollars to individuals or families who used the crossing frequently. A devout churchman, he advertised before Christmas that all who wished to attend Christmas services at his church from points in New Hampshire could pass the bridge free of charge.

In 1865, when Mr. Tucker died, ownership of the bridge passed to his grandson, Edward H. Green, who purchased the interest of the heirs. In that year Mr. Green married Hetty Robinson, who was later known as Hetty Green, the richest woman of the country. Tradition says that she took great pleasure and pride in the fact that she could cross the bridge without paying toll. From Edward Green the latticed bridge passed to paper-making interests in 1880, and in 1902 to a power company. In 1904 it was purchased by the towns of Walpole, N.H., and Rockingham, Vt., and made a free bridge and so continued until it failed from neglect of a wall block, which injured one truss beyond repair.

Thus the famous Tucker Toll Bridge stood for 90 years, until it was replaced by a beautiful two-span concrete arch structure, designed and built to carry the loads imposed by modern high-speed motor traffic. This bridge was opened to traffic on April 16, 1931. The stone arched structure shown in the photograph on the upstream side of the concrete bridge is a railroad bridge built in 1899 to replace the original timber structure, constructed about 1851. The concrete bridge occupies the same location as Tucker's Bridge, and was built for the State Highway Commission of New Hampshire. After its erection a public-minded citizen of Walpole, Mr. Vilos, paid the cost of the bridge, and in recognition of this generosity it is now known as the Vilos Bridge. The new bridge, a monument to mark the passing of two historical masterpieces of wooden bridge construction, will continue to serve the traveling public for many years to come.

Public Works and the Construction Industry as a Career

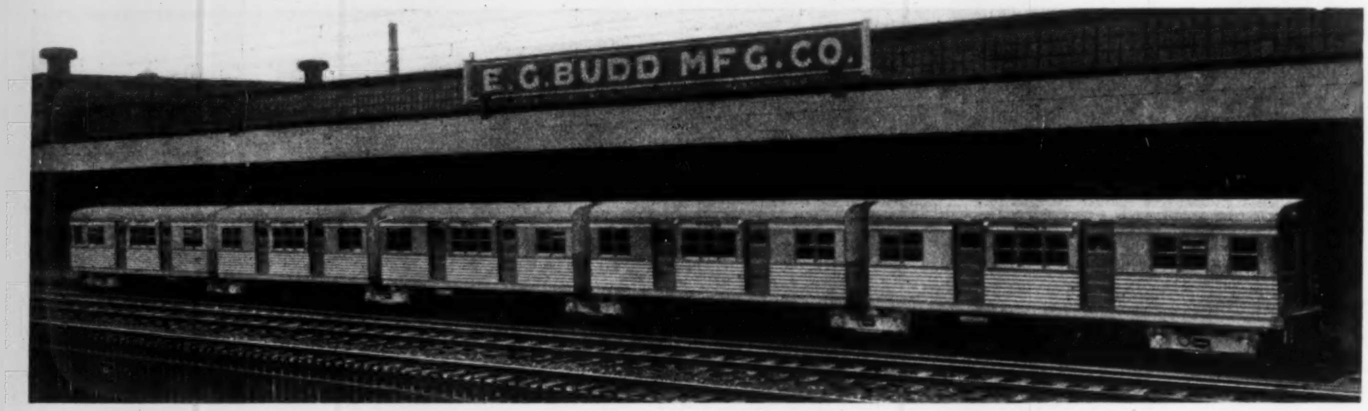
The College of Engineering at New York University announces a new curriculum in the senior civil engineering year, designed to train students especially for a career in public works and the construction industry. The civil engineer has always supplied the greatest number of technically trained men to the public service. During the course of the depression it has become clear that planned public works activities must be relied upon increasingly as a regulator of unemployment and a balance wheel for industry, business, and the public welfare.

In the planning, execution, and administration of large public works and construction projects, the place of the civil engineer is growing more and more important. The opportunity for civil engineers particularly trained in the broader aspects of these activities has caused the College of Engineering to introduce a public works and construction option in the senior civil engineering year. In connection with this option, believed to be the first of its kind in this country, some of the more highly specialized subjects ordinarily given in the senior year will be condensed or omitted to make room for five new courses. These are: (1) engineering economics and finance; (2) contractor's organization and equipment; (3) construction codes and labor problems; (4) industrial organization; and (5) public works organization and administration.

These new courses will cover the broad economic and social principles on which public works should be based, and describe the practical aspects of their organization and functioning. The relation of the civil engineer to the construction industry will be stressed. For the present these courses will be open only to those seniors who have had honor grades during their three previous years. To a large extent the new courses will be given as coordinated lectures by prominent practicing engineers, economists, and public officials who have interested themselves in the objects of this undertaking.

A new era has arrived, an epoch of different concepts of the relation of public to private activity, a time in which public works as planned, constructed, and administered by the civil engineer have become part of an emergency program widely accepted and likely to continue in modified form as a permanent national policy.

The construction codes are designed to create a better integrated and a well organized construction industry. These developments, regarded as permanent in principle, though likely to change in form, offer new and challenging opportunities and responsibilities to the civil engineer.



Brooklyn-Manhattan Transit Corporation's Five-Section Car of Stainless Steel
Built by the Edward G. Budd Manufacturing Company, Philadelphia

Light-Weight Subway Unit

A five-section articulated car of stainless steel, powered with a 70-hp motor on each of 12 axles, was placed in experimental operation July 16, 1934, by the Brooklyn-Manhattan Transit Corporation. Its capacity is 180 passengers seated and 460 standing. The car, similar in construction to the Burlington Zephyr, can be accelerated at the rate of 5 mph per sec and is equipped for eddy-current braking.

NEWS OF ENGINEERS

From Correspondence and Society Files

C. F. Wertz has been transferred from the Chicago office of the Nichols Engineering and Research Corporation to the main office of the same organization at 40 Wall Street, New York, N.Y.

T. T. Knappen has been transferred from the position of area engineer in the U. S. Engineer Office at Cairo, Ill., to that of chief of the Engineering Division of the Zanesville Engineer District of the same office, with headquarters in Zanesville, Ohio.

Ralph A. Sturgeon recently severed his connection with the U. S. Engineer Office at Osceola, Ark., to accept an appointment as agricultural engineer in the Soil Erosion Service of the U. S. Department of the Interior, with offices at Conway, Ark.

William R. Franklin has accepted a position in the Construction and Equipment Department of Montgomery Ward and Company, of Chicago, Ill.

Thomas G. Croom is now Assistant Director of Community Sanitation for the Indiana Division of Public Health, with headquarters in Indianapolis, Ind.

Harry A. Cox, formerly designing engineer for Thebo, Starr, and Anderton, Inc., of San Francisco, Calif., has accepted a connection with the State of California Department of Public Works in the capacity of structural engineer. His office is in Los Angeles, Calif.

Harry Larson has taken the position of resident engineer for the Federal Emergency Administration of Public Works, with headquarters in Iowa City, Iowa.

Ralph S. McLean is now employed as a seismologist by the U. S. Coast and Geodetic Survey. His headquarters are in Los Angeles, Calif.

A. R. Nieman, formerly a designer for the Stone and Webster Engineering Corporation, is now chief project engineer of the Edwards County (Illinois) Civil Works Administration, with offices in Albion, Ill.

Ralph T. Taylor has resigned as engineer and general construction superintendent for W. A. Taylor and Sons, of Pasadena, Calif., to become senior structural engineer in the Division of Architecture of the State of California Department of Public Works. His office is in Los Angeles, Calif.

Gifford M. Irwin has been appointed city engineer and water commissioner of Victoria, B.C., after filling these positions in an acting capacity for the past year and a half.

Harold C. Hart, formerly with the Water Bureau of the Metropolitan District of Hartford County, Connecticut, has accepted an appointment as a junior engineer with the U. S. Bureau of Reclamation in Denver, Colo.

W. W. Studdert has been promoted from the position of observer on triangulation for the U. S. Coast and Geodetic Survey to that of chief of party on precise levels in the same organization. His headquarters are in Williamsport, Pa.

Carroll Tracy Morton, formerly an engineer with the Healy-Tibbitts Construction Company, of San Francisco, Calif., has accepted a connection with the California Redwood Association, of Los Angeles, Calif.

Joseph J. Rosa has severed his connection with the U. S. Coast and Geodetic Survey to accept a position in the U. S. Engineer Office, Second District, with headquarters in New Orleans, La.

P. E. Barber, Jr., formerly a transitman for the Missouri Pacific Lines, is now in the employ of the Humble Oil and Refining Company, of Houston, Tex.

Donald P. Barnes has resigned his position as assistant engineer for the Metropolitan Water District of Southern California to accept a connection in a similar capacity with the U. S. Bureau of Reclamation in Denver, Colo.

E. E. Grossman is now employed by the General Realty and Utility Company, of New York, N.Y. He was formerly treasurer for T. J. Doherty, Inc., of the same city.

R. H. Baker, who was formerly Commissioner of the Tennessee State Department of Highways and Public Works, has accepted a position as sales manager of the Pennsylvania-Dixie Cement Corporation, with offices in Chattanooga, Tenn.

Raymond L. Moore has severed his connection with the Ohio State Department of Highways to enter the Engineering Service Division of the Tennessee Valley Authority, with headquarters in Decatur, Ala.

Walter M. Smith, who has been serving as acting chief engineer of the Division of Waterways, Department of Public Works and Buildings, State of Illinois, for the past year, has been appointed chief engineer of that division.

H. Alden Foster recently resigned as associate hydraulic engineer of the U. S. Geological Survey to accept the position of resident engineer for the firm of Parsons, Klapp, Brinckerhoff and Douglas, of New York, N.Y., consulting engineers for the Platte Valley Public Power and Irrigation District. His headquarters are in North Platte, Nebr.

Mace H. Bell, Jr., is now with the firm of Harrington and Cortelyou, consulting engineers of Kansas City, Mo. He was formerly a draftsman for the Mt. Vernon Bridge Company, of Mt. Vernon, Ohio.

Francis G. Christian has been transferred from the position of assistant chief of the General Engineering Division of the U. S. Engineer Office, in Vicksburg, Miss., to that of chief of the Planning Section and assistant to the chief of the General Engineering Division of the U. S. Engineer Office at Zanesville, Ohio.

George S. Siekielski has accepted a connection with the Tennessee Valley Authority in the capacity of erosion engineer, with offices in Knoxville, Tenn.

D. Grant Mickle, formerly with the Massachusetts Department of Public Works, has accepted a position with the U. S. Bureau of Public Roads as traffic supervisor on the Arkansas Traffic Survey. His headquarters are at Russellville, Ark.

Changes in Membership Grades

Additions, Transfers, Reinstatements, Deaths, and Resignations

From June 10 to July 9, 1934, Inclusive

Additions to Membership

Adair, George Parke (Jun. '34), Field Engr., Erection Dept., McClintic-Marshall; 45 Clark Ave., Lynbrook, N.Y.

Basta, Rudolph Francis (Jun. '33), Timekeeper and Instrumentman, Ryan Const. Co. (Res., 1808 Emmet St.), Omaha, Nebr.

Beffa, Theodore Albin (Assoc. M. '34), Engr., City of St. Louis, 809 South 6th St., St. Louis, Mo.

Brown, Walter Emerson (M. '34), With Delaware River Joint Comm., Camden (Res., 340 East Park Ave., Haddonfield), N.J.

Brumund, Gerry Henry (Jun. '34), Junior Engr., U. S. Geological Survey, Box 333, Yosemite National Park, Calif.

Burr, Howard Edwin (Assoc. M. '34), Insp., Erie R. R., Box 68, Fair Lawn, N.J.

Cozzens, Howard Francis (Assoc. M. '34), County Surv. and Supt. of Highways and Bridges, Monterey County, Box 426, Salinas, Calif.

Deering, Mark Fay (Assoc. M. '34), Structural Engr., S. B. Barnes (Res., 515 North Poinsettia Pl.), Los Angeles, Calif.

Emerson, Merton Leslie (M. '34), Cons. Engr., 110 State St., Boston, Mass.

Fry, George Lambert (Assoc. M. '34), Asst. Engr., Plans and Surveys, State Highway Comm., Little Rock, Ark.

Gibson, Alexander McKay (Jun. '34), Senior Draftsman, State Dept. of Highways, Upper Darby (Res., 5790 Haddington St., West Philadelphia), Pa.

Harth, Wilfrid Henry (Jun. '34), Junior Officer, U. S. Coast and Geodetic Survey, Box 468, Miami, Fla.

Haygard, Albert (Assoc. M. '34), Engr., U. S. Bureau of Public Roads, Front Royal, Va.

Jensen, Emmanuel Tranberg (Jun. '34), Research Asst., Iowa Eng. Experiment Station (Res., 218 North Hyland Ave.), Ames, Iowa.

Jopson, Leslie Carlysle (Assoc. M. '34), Res. Engr. and Water Master, State Div. of Water Resources, Box 1079, Sacramento, Calif.

Kendall, William Hersey (Jun. '33), Asst. Eng. Corps., P. R. R., 928 Main St., Wellsville, Ohio.

Knapp, Ronald Clarence (Assoc. M. '34), Bridge Designer, Bridge Div., State Highway Dept. (Res., 2209 Washington St.), Olympia, Wash.

Louison, Ben Howe (Assoc. M. '34), Structural Engr., Palmer & Turner, 495 Honan Rd., Shanghai, China.

Louree, Charles Harold (Jun. '33), With J. F. Pritchard & Co., Dwight Bldg., Kansas City, Mo.

McManamna, Theodore Louis (Assoc. M. '34), Field Engr. and Mgr., International Water Supply, Ltd., 25 King St., West (Res., 212 Duplex Ave.), Toronto, Ont., Canada.

Mott, Guibert Allen (Jun. '33), Stoddard, N.H.

Murdichian, Karmy Karnig (Jun. '34), Draftsman, Gibbs & Hill, Pennsylvania Station, New York (Res., 1071 Fulton St., Brooklyn), N.Y.

Philip, Sidney (Jun. '33), 1546 Forty-seventh St., Brooklyn, N.Y.

Pineo, Charles Stanley (Assoc. M. '34), Asst. Engr., Panama Canal, Box 647, Balboa, Canal Zone.

Prisadsky, Vladislav Alexander (Jun. '34), 9006 Thirty-fifth Ave., N. E., Seattle, Wash.

Pruden, Worrell Franzoni (Jun. '34), Chainman, Los Angeles County Flood Control Dist.; $920^{1}/_{2}$ Belmont Ave., Long Beach, Calif.

Ralston, Harry James (Jun. '34), Care, A. M. Burleigh, High St., Hampton, N.H.

Rasul, Mir Ghulam (Jun. '34), Naid Kadal, Srinagar, Kashmir, India.

Savage, Richard Ewart (Assoc. M. '33), Asst. Engr., California Railroad Comm., State Bldg., San Francisco (Res., 1556 James Ave., Redwood City), Calif.

Swanton, Walter Frederick (Jun. '34), Junior Engr., Bureau of Reclamation, U. S. Dept. of Interior, Customhouse, Denver, Colo.

TOTAL MEMBERSHIP AS OF JULY 9, 1934

Members	5,750
Associate Members	6,284
Corporate Members	12,034
Honorary Members	17
Juniors	3,155
Affiliates	106
Fellows	4
Total	15,316

Membership Transfers

Butler, Merrill (Assoc. M. '23; M. '34), Deputy City Engr., Bureau of Eng. (Res., 1563 South Ogden Drive), Los Angeles, Calif.

Cone, Russell Glenn (Jun. '22; Assoc. M. '26; M. '34), Res. Engr., Golden Gate Bridge and Highway Dist., for Strauss Eng. Corporation, Box 47, Presidio Station, San Francisco, Calif.

Green, John Wesley (Assoc. M. '29; M. '34), Associate Bridge Engr., State Div. of Highways, San Francisco-Oakland Bay Bridge, 500 Sansome St., San Francisco, Calif.

Hussey, Harold Dudley (Assoc. M. '24; M. '34), Designing Engr., Am. Bridge Co., 71 Broadway, New York (Res., 146 North High St., Mount Vernon), N.Y.

Kindsvater, Emil Fred (Jun. '24; Assoc. M. '34), Engr., Phillips Petroleum Co. (Res., 1343 Keeler Ave.), Bartlesville, Okla.

Montz, John McGill (Assoc. M. '24; M. '34), Associate Prof., Civ. Eng. Dept., Ohio State Univ. (Res., 172 Orchard Lane), Columbus, Ohio.

Moyer, Tilghman Huber (Assoc. M. '21; M. '34), Pres. and Gen. Mgr., Tilghman Moyer Co. (Res., 116 North 15th St.), Allentown, Pa.

Noble, John Henry Neidlinger (Jun. '25; Assoc. M. '34), Engr., Wire Rope Div., John A. Roebling's Sons, Co., Trenton, N.J. (Res., 145 West Maple Ave., Morrisville, Pa.)

Pajot, Clayton James Joseph (Jun. '32; Assoc. M. '34), Instr., Eng. Mechanics, Univ. of Detroit, McNichols Rd. and Livernois Ave., Detroit, Mich.

Schlemmer, Oren Henry (Jun. '27; Assoc. M. '33), 1519 Dexter Ave., Ann Arbor, Mich.

Unger, George Frederick (Assoc. M. '21; M. '34), Chf. Engr., Niagara Frontier State Park Comm., State Office Bldg., Buffalo, N.Y.

Wilson, William Sidney (Jun. '30; Assoc. M. '34), Junior Engr., Met. Water Dist. of Southern California; 3730 First Ave., San Diego, Calif.

RESIGNATIONS

Baldwin, Ernest Wood, Assoc. M., resigned June 8, 1934.

Bender, Harry, Jun., resigned June 13, 1934.

Brown, Wallace Francis, Assoc. M., resigned June 13, 1934.

Clausen, Robert Eskilson, Jun., resigned July 2, 1934.

Crandall, Fred Nelson, Jun., resigned June 13, 1934.

Franks, Fred Lester, Assoc. M., resigned June 8, 1934.

Kleinschmidt, Robert Baumgartner, Jun., resigned July 5, 1934.

Macbeth, James Alexander Duff, Assoc. M., resigned June 21, 1934.

Senior, Jack, Assoc. M., resigned July 2, 1934.

DEATHS

Baker, Joseph Anthony. Elected Assoc. M., Nov. 26, 1923; died May 19, 1934.

Bogardus, Jared Sperry. Elected M., June 1, 1920; died June 19, 1934.

Eldredge, Charles Guy. Elected Affiliate, June 3, 1915; died Nov. 9, 1933.

Foster, Walter Linder. Elected Assoc. M., May 31, 1916; M., Jan. 20, 1922; died May 23, 1934.

Fuller, George Warren. Elected Assoc. M., Mar. 1, 1899; M., May 31, 1904; died June 15, 1934.

Gore, William. Elected M., Jan. 18, 1926; died June 6, 1934.

Grunsky, Carl Ewald. Elected M., Oct. 5, 1898; died June 9, 1934.

Harris, Van Alen. Elected Jun., Nov. 6, 1894; Assoc. M., Oct. 5, 1898; M., Sept. 6, 1904; died May 31, 1934.

McCarrel, Arthur Hicks. Elected M., Jan. 13, 1930; died June 12, 1934.

McCarty, Richard Justin. Elected M., Dec. 4, 1895; died June 16, 1934.

Nixon, Courtland. Elected Assoc. M., Nov. 1, 1905; died June 30, 1934.

Ritter, Louis E. Elected M., Oct. 4, 1905; died July 3, 1934.

Rosenthal, Albert. Elected Jun., Jan. 2, 1894; Assoc. M., Nov. 6, 1901; died April 25, 1934.

Savage, Hiram Newton. Elected Assoc. M., Mar. 7, 1934; M., Oct. 7, 1896; died June 24, 1934.

Sessions, Edson Oliver. Elected Assoc. M., July 9, 1912; date of death unknown.

Smith, Albert Orange. Elected Jun., May 2, 1905; Assoc. M., Feb. 2, 1909; died June 30, 1934.

Veeder, Herman Greig, Jr. Elected Jun., Oct. 1, 1928; died June 11, 1934.

Weigand, Edwin John. Elected M., Feb. 23, 1932; died June 12, 1934.

Men Available

These items are from information furnished by the Engineering Societies Employment Service, with offices in Chicago, New York, and San Francisco. The Service is available to all members of the contributing societies. A complete statement of the procedure, the location of offices, and the fee is to be found on page 85 of the 1934 Year Book of the Society. To expedite publication, notices should be sent direct to the Employment Service, 31 West 39th Street, New York, N.Y. Employers should address replies to the key number, care of the New York Office, unless the word Chicago or San Francisco follows the key number, when the reply should be sent to the office designated.

CONSTRUCTION

Concrete Technician; Assoc. M. Am. Soc. C.E.; technical education; married; 8 years construction experience, specializing in concrete design, control, charge of inspection force and concrete testing laboratory. Qualified to take charge of concrete field laboratory, concrete inspection force, making special investigations in concrete, and preparing reports. Location immaterial. Available 1 months notice. C-8022.

DESIGN

Civil Engineer; Assoc. M. Am. Soc. C.E.; 38; married; graduate; state licenses; 10 years experience, surveying, design, and construction. Railroad, highway, parkway, harbor, and tall building projects. Completely familiar with the design of rigid-frame bridges, reinforced concrete or steel, single or multiple-span, square or skew. Available immediately. D-1496.

Structural and Municipal Engineer; Assoc. M. Am. Soc. C.E.; graduate C.E.; 15 years experience. Experienced designer, estimator, and superintendent, principally architectural engineering and sewage disposal. Competent to limited extent on mechanical installations, heating and refrigeration, pumps and conveying machinery. Also considerable experience in the design of heavy railroads, movable bridges, and cranes. Will go anywhere, including abroad. C-2701.

Civil Engineer; Assoc. M. Am. Soc. C.E.; graduate C.E.; 33; married; 10 years experience in the design of reinforced concrete and structural steel bridges, steel plants, blast furnace, skip bridges, indeterminate structures, heavy foundations. Writes and speaks German and French perfectly. Suitable man for foreign work. Available at once. Location and salary secondary. First-class references. D-2019.

Structural Designer and squad boss, on steel and concrete structures, desires position; Assoc. M. Am. Soc. C.E.; age 34; experience covers mill buildings, bins, towers, conveyor bridges for cement plants; also gas and coke oven plants, heavy railroad viaduct, elevated streets, tunnels, sewer lines, pile and caisson foundations. Available immediately. C-8644.

EXECUTIVE

Civil Engineer; M. Am. Soc. C.E.; with 20 years experience as construction superintendent, resident engineer, designer, surveyor, inspector, draftsman; desires position. Experienced in the design and construction of steam, hydro electric, and cement plants, irrigation works, railroad structures, sewers and pavements. High-class references. D-535.

Graduate Civil Engineer; Assoc. M. Am. Soc. C.E.; 37; married; 7 years experience from draftsman to chief draftsman, steel and reinforced concrete structures; 8 years principal assistant bridge engineer in state highway department, responsible charge of office employing 30 designers and resident engineers on construction, covering all types of highway bridges. Available immediately. D-707.

Civil and Sanitary Engineer; Assoc. M. Am. Soc. C.E.; 35; married; B.S. in sanitary engineering, Massachussetts Institute of Technology; licensed 12 years; varied and extensive experience in municipal field; executive ability. Desires position as city manager; charge of sales or purchasing; or similar position where ability will be recognized. Location immaterial. Excellent references. C-84.

Civil Engineer; M. Am. Soc. C.E.; 28 years experience, covering investigations and reports, laboratory testing of materials, organization, design, reinforced concrete construction. Specialist in checking coast erosion by sea-defense works based on sound and proved principle. Speaks French, Spanish. Location immaterial. Available now. D-2989.

Graduate Civil Engineer, 1922; Jun. Am. Soc. C.E.; age 30; 3 years construction experience; 7 years as engineer in technical department of two cement companies. Duties consisted of consultation with customers, cooperation with engineers and architects on concrete problems, promotion of products, design of mixtures for ready-mix concrete, office correspondence. C-370.

Civil Engineer; Assoc. M. Am. Soc. C.E.; 34; married; licensed professional engineer; desires position in some branch of the aviation industry. Experience includes design, construction, and maintenance of airports, hangars, and all ground equipment used in air-line operation. Available one months notice. Location immaterial. D-3180.

Technical Graduate; Assoc. M. Am. Soc. C.E.; master of two trades; 30 years experience in construction work consisting of structural steel and monumental buildings. Also 7 years as head of safety engineering in largest industrial plant in America and assistant in personnel work. Will go any place in the United States. Salary open. C-4259.

Structural and Civil Engineer; Assoc. M. Am. Soc. C.E.; graduate of University of California and Wilson Engineering Corporation; 10 years on the design and construction of industrial buildings and structures and the installation of plant equipment, 7 of which involved full responsibility; 4 years of reclamation, highway, and geodetic survey experience. Immediately available and location immaterial. C-6910.

Graduate Engineer; Assoc. M. Am. Soc. C.E.; Massachusetts Institute of Technology, 1921; 36; married. Experience: sales, tests and reports, machine and construction designs, estimating, supervision, purchasing, and specifications. Writes and speaks French fluently. Conscientious and ambitious man available on short notice for responsible position. B-5822.

Structural Engineer Executive; M. Am. Soc. C.E.; 46; married; registered in Pennsylvania. Connected with engineering management and sales, steel fabricating, and general construction business past 25 years, in western Pennsylvania, W.Va., and Ohio districts. Desires opportunity to communicate in detail with persons interested in having man supervise whole or part of any organization most efficiently. C-5095.